

**FOUNDATION INVESTIGATION AND DESIGN REPORT
BUG RIVER BRIDGE REPLACEMENT
HIGHWAY 105, RED LAKE DISTRICT, ONTARIO
G.W.P. 6942-10-00, SITE 41N-2**

Geocres Number: 52K-8

Report to

GENIVAR

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October 16, 2012
File: 19-5308-40

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the location of a proposed bridge replacement carrying Highway 105 over Bug River. The existing Bug River bridge is approximately 9.8 km south of Highway 125 in the Red Lake District, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile and sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Information on subsurface conditions contained in a previous foundation report for this site was also assessed during preparation of this report. The title of this reference report is listed as follows:

- Report on a Foundation Investigation for The Bug River Bridge, Highway No. 105, Red Lake District, Ontario, dated June 6, 1956 by Racey, MacCallum & Associates Ltd. (Reference 1).

The previous report is included in Appendix C for reference.

Thurber carried out the investigation as a sub-consultant to GENIVAR, under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0027.

2 SITE DESCRIPTION

The Bug River Bridge is located on Highway 105, approximately 9.8 km south of Highway 125 and 25 km north of Snake Falls, in the Red Lake District, Ontario.

Highway 105 is a two-lane paved road. The existing structure consists of a six-span bridge with a concrete and timber deck which was built in 1958. Each span is approximately 6 m long, for a total bridge length of 37 m. The width of the existing bridge is 9.8 m. The concrete wing walls, deck, curbs, approach slabs, and barrier walls were replaced in 1989.



At this location, Bug River flows from south to north into Gullrock Lake. As noted on the preliminary General Arrangement drawing (GA) provided by GENIVAR, the water level in the Bug River was measured at elevation 355.7 m in May 2011.

The lands immediately surrounding the bridge site consist primarily of forested areas. Immediately to the northwest of the bridge, the land consists of a low lying area with swamp vegetation.

Selected photographs in Appendix D show the general nature of the surrounding land and the existing bridge structure. Photos of the site indicate presence of rock fill on the forward and side slopes below the existing abutments. It is not confirmed if this rockfill is for erosion control purposes or whether the embankments contain rockfill.

The site is located within the Red Lake District, which is underlain by Archean rocks of the Superior Province of the Canadian Shield. Based on bedrock geology maps published by the Ontario Geological Survey, the site is located in an area that is underlain by felsic to intermediate metavolcanic rocks. Locally, the overburden consists primarily of silt and sand deposits.

3 SITE INVESTIGATION AND FIELD TESTING

The approximate locations of the boreholes drilled for the current investigation are shown on the attached Borehole Locations and Soil Strata Drawing included in Appendix H. The boreholes were drilled behind the existing abutment locations. The latest General Arrangement (GA) drawing shows that the proposed abutments are located about 3.0 m to 4.0 m in front of the existing abutments. The borehole locations were marked in the field and utility clearances were obtained prior to drilling.

The site investigation and field testing for this project were carried out from August 7 to 9, 2011 and consisted of drilling and sampling six boreholes (identified as BUG-01 to BUG-06) and performing one Dynamic Cone Penetration Test (DCPT). All boreholes, as well as the DCPT, were advanced through the highway embankments at the existing bridge site. Boreholes BUG-01 and BUG-06 were drilled at the north and south approaches, respectively and were terminated at depths of 10.8 m and 11.3 m (elevations 347.9 and 347.6). Boreholes BUG-02 and BUG-03 were drilled near the north abutment while Boreholes BUG-04 and BUG-05 were drilled near the south abutment. Boreholes BUG-02 to BUG-05 were advanced within the overburden to depths ranging from 12.9 m to 14.0 m (elevations 345.9 to 344.8). Bedrock was proved in Boreholes BUG-02 and BUG-05 by NQ size diamond coring. Borehole BUG-02 was advanced 2.7 m into bedrock and terminated at a depth of 16.4 m (elevation 342.2). Borehole BUG-05 was advanced 3.7 m into bedrock and terminated at a depth of 17.7 m (elevation 341.2). Boreholes BUG-03 and BUG-04 were terminated upon auger refusal on probable bedrock or boulders at 13.4 m and 12.9 m depth, (elevations 345.3 and 345.9), respectively.

Borehole BUG-03 was supplemented by a DCPT conducted adjacent to the borehole. The DCPT was conducted from 1.5 m to 13.3 m depth (elevations 357.2 to 345.4). The DCPT was terminated upon refusal.

Three boreholes (BH 1, BH 2, and BH 3) were advanced at this site for the 1956 foundation investigation (Reference 1). The borehole logs of the previous investigation are included in Appendix C. These boreholes were advanced to depths of approximately 13.7 m to 16.3 m. Boreholes BH 1 and BH 2 were drilled on the north and south sides of the river and BH 3 was drilled through the bridge deck in the middle of the river. There is insufficient data in the 1956 foundation report to determine the exact locations and ground elevations of these boreholes. However, the subsurface conditions encountered in these 3 boreholes appear to correlate well with the subsurface conditions encountered in the 6 boreholes drilled for the current investigation.

The drilling was carried out from the highway grade using a CME 75 truck-mounted drill rig. A combination of hollow-stem augers, NW casing and NQ coring methods were used to advance the boreholes. Overburden samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT).

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil samples for transport to Thurber's laboratory for further examination and testing.

All rock cores were logged, and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

Groundwater conditions were observed in the open boreholes during the drilling operations. One standpipe piezometer, consisting of 19 mm diameter PVC pipe with a slotted screen, was installed at each abutment and enclosed in filter sand to permit longer term groundwater level monitoring. The boreholes were backfilled with bentonite holeplug in general accordance with O.Reg. 903 upon completion. The locations and completion details of the boreholes and piezometers are presented in Table 3.1.

Table 3.1 – Borehole Abandonment Details

Location	Borehole	Piezometer Tip Depth/ Elevation (m)	Abandonment Details
North approach	BUG-01	None installed	Backfilled with bentonite holeplug from 10.8 m to 1.9 m, sand from 1.9 m to 0.2 m, then asphalt to surface.
North abutment	BUG-02	None installed	Backfilled with bentonite holeplug from 16.4 m to 1.8 m, auger cuttings from 1.8 m to 0.6 m, sand from 0.6 m to 0.3 m, then asphalt to surface.
	BUG-03	13.4 / 345.3	Piezometer with 1.5 m slotted screen installed with sand filter to 11.1 m, bentonite holeplug from 11.1 m to 2.6 m, auger cuttings from 2.6 m to 0.6 m, sand from 0.6 m to 0.3 m, then cement to surface. Flushmount casing protector installed.
South Abutment	BUG-04	12.2 / 346.6	Piezometer with 1.5 m slotted screen installed with sand filter from 12.2 m to 10.0 m, bentonite holeplug from 10.0 m to 2.2 m, auger cuttings from 2.2 m to 0.6 m, sand from 0.6 m to 0.3 m, then cement to surface. Flushmount casing protector installed.
	BUG-05	None installed	Backfilled with bentonite holeplug from 17.7 m to 1.6 m, auger cuttings from 1.6 m to 0.3 m, then cement to surface.
South approach	BUG-06	None installed	Backfilled with bentonite holeplug from 11.3 m to 1.5 m, auger cuttings from 1.5 m to 0.1 m, then concrete to surface.

The piezometers will be decommissioned in accordance with O. Reg. 903 prior to the end of 2012.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to gradation analysis (hydrometer and sieve) and Atterberg Limits tests, where appropriate. The results of these tests are summarized on the Record of Borehole sheets included in Appendix A and are presented on the figures included in Appendix B.

Point load tests were carried out on selected samples of intact bedrock upon arrival at the laboratory to assist in evaluation of the compressive strength of the bedrock. Results of point load tests on the rock core samples are included in Appendix B and on the Record of Borehole sheets in Appendix A (as average unconfined compressive strength per run).

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil and rock stratigraphy are presented in these sheets and on the “Borehole Locations and Soil Strata” drawing included in Appendix H. An overall description of the stratigraphy is

given in the following paragraphs. However, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions.

In general terms, the overburden soil stratigraphy encountered at this site consists of sand and gravel fill overlying native layers of sand and sandy silt to silty sand. Silty clay was encountered below the fill and above the sandy silt to silty sand in both approach boreholes. A 0.6 m to 1.4 m thick layer of organic clayey silt/silty clay was encountered at the south abutment. The overburden is underlain by highly to moderately weathered metamorphic bedrock of gneissic structure. More detailed descriptions of the individual strata are presented below.

5.1 Pavement structure

Pavement structure was encountered in all the boreholes drilled at this site. The boreholes were drilled through the existing Highway 105 lanes. The pavement structure consists of approximately 125 mm to 150 mm of asphalt overlying granular fill.

5.2 Sand and Gravel Fill

Granular fill consisting of sand and gravel containing trace silt and clay and occasional cobbles was encountered below the asphalt in all the boreholes. The thickness of the granular fill ranges from 2.8 m to 3.8 m, with the base of the granular fill encountered at depths ranging from 2.9 to 4.0 m (elevations 354.9 to 356.0).

SPT N-values recorded in the sand and gravel fill ranged from 3 to 49 blows for 0.3 m penetration, indicating a loose to dense relative density. Typically, the higher SPT 'N' values were recorded near the ground surface.

The moisture contents of samples of the sand and gravel fill ranged from 2% to 22%. Typically, the moisture content of the granular fill was less than 5%.

Two samples of the granular fill were selected for laboratory gradation analysis, the results of which are summarized below. These results are also presented on the Record of Borehole sheets in Appendix A and the grain size distribution curves for these samples are plotted on Figure B1 of Appendix B.

Soil Particles	Percentage (%)
Gravel	36
Sand	57 to 59
Silt and Clay	5 to 7

As indicated earlier, rockfill is visible on the forward and side slopes below the existing abutments. It is not confirmed if this rockfill is for erosion protection purposes or whether the embankment below and in front of the abutments were constructed with rockfill. No boreholes were drilled in these areas, where rockfill is visible in the sideslopes below the

abutments. It must be recognized that embankment fills are heterogeneous in nature and may contain obstructions such as boulders or rockfill.

5.3 Silty Clay

Silty clay was encountered below the granular fill in Boreholes BUG-01, BUG-04, and BUG-06. The silty clay contains trace sand and occasional rootlets. The thickness of the silty clay ranges from 0.9 m to 3.1 m, with the base of the silty clay encountered at depths of 4.3 to 6.1 m (elevations 352.6 to 354.5).

SPT N-values recorded in the silty clay ranged from 5 to 7 blows for 0.3 m penetration, indicating a firm consistency.

The moisture contents of samples of the silty clay ranged from 22% to 32%.

Two samples of the silty clay were selected for laboratory gradation analysis, the results of which are summarized below. These results are also presented on the Record of Borehole sheets in Appendix A and the grain size distribution curves for these samples are plotted on Figure B2 of Appendix B. One sample of the silty clay also underwent Atterberg Limits testing, the results of which are included on the Record of Borehole sheets and plotted on Figure B7 of Appendix B. These results are also presented below.

Soil Particles	Percentage (%)
Gravel	0
Sand	5
Silt	38 to 52
Clay	43 to 57

Index Property	Percentage (%)
Liquid Limit	58
Plastic Limit	22
Plasticity Index	36

Results of the Atterberg Limits tests indicate that the silty clay is of high plasticity with a group symbol of CH.

5.4 Sand

Native brown to grey sand containing trace to some gravel and trace silt and clay was encountered in all of the boreholes, with the exception of Borehole BUG-01, at the depths and elevations indicated in Table 5.1.

Table 5.1 – Depths and Elevations of Native Sand

Foundation Unit	Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
North abutment	BUG-02	3.0 to 5.0	355.7 to 353.7	2.0
		10.0 to 13.7	348.7 to 345.0	3.7
South abutment	BUG-03	3.7 to 5.5	355.0 to 353.2	1.8
		8.5 to 13.4 (borehole termination depth)	350.2 to 345.3	4.9
South abutment	BUG-04	5.5 to 12.9 (borehole termination depth)	353.4 to 345.9	7.4
		3.0 to 5.2	355.9 to 353.7	2.2
South Approach	BUG-05	6.6 to 14.0	352.3 to 344.8	7.4
		6.1 to 11.3 (borehole termination depth)	352.8 to 347.6	5.2

Trace gravel and occasional wood fragments were noted in the upper sand layer.

Occasional cobbles and rock fragments were encountered within the lower sand layer.

SPT ‘N’ values measured in the sand ranged from 1 to 31 blows per 0.3 m of penetration, indicating a very loose to dense relative density. Typically, ‘N’ values recorded in the sand were between 4 and 15 blows for 0.3 m penetration, indicating a loose to compact relative density. A SPT ‘N’ value of 100 blows for 0.1 m penetration was recorded in Borehole BUG-05 at a depth of 13.1 m due to the presence of a probable boulder or bedrock.

The natural moisture contents of samples of the sand ranged from 10% to 21%.

Selected samples of the sand underwent laboratory grain size analysis testing, the results of which are summarized below. These results are also presented on the Record of Borehole sheets in Appendix A and the grain size distribution curves for these samples are plotted on Figure B4, Appendix B.

Soil Particles	Percentage (%)
Gravel	0 to 26
Sand	62 to 95
Silt	19
Clay	2
Silt and Clay	3 to 12

5.5 Layer of Organic clayey silt/silty clay

A dark brown layer of organic clayey silt/silty clay containing wood fragments, and trace rootlets was encountered below the silty clay in Borehole BUG-04 and below the native sand in Borehole BUG-05 at the south abutment.

The thickness of the organic clayey silt/silty clay layer was 0.6 m in Borehole BUG-04 and 1.4 m in Borehole BUG-05. The base of the layer of organic clayey silt/silty clay was at depths of 5.5 m and 6.6 m (elevations 353.4 and 352.3) in Boreholes BUG-04 and BUG-05, respectively.

SPT N-values of 0 to 5 blows for 0.3 m penetration were recorded in the layer of organic clayey silt/silty clay, indicating a very soft to firm consistency.

The moisture content of two samples collected from the layer of organic clayey silt/silty clay were 70% and 74%.

5.6 Sandy Silt

A layer of sandy silt was encountered below the layer of sand in Boreholes BUG-02 and BUG-03. The sandy silt is grey and contains trace to some clay. The sandy silt layer was 1.6 m thick in Borehole BUG-02 and 3.0 m thick in Borehole BUG-03. The base of the sandy silt was at depths of 6.6 m and 8.5 m (elevations 352.2 and 350.2) in Boreholes BUG-02 and BUG-03, respectively.

SPT 'N' values recorded in the sandy silt layer ranged from 2 to 8 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The moisture content of samples of the sandy silt ranged from 18% to 27%.

One sample of the sandy silt underwent laboratory gradation analysis. The grain size distribution curve for this sample is plotted on Figure B5 of Appendix B. The results of the gradation analysis are summarized as follows and are presented on the Record of Borehole sheets in Appendix A.

Soil Particles	Percentage (%)
Gravel	0
Sand	22
Silt	71
Clay	7

5.7 Silty Sand

Grey silty sand containing trace clay and trace gravel was encountered in Boreholes BUG-01, BUG-02, and BUG-06 at the depths and elevations indicated in Table 5.2.

Table 5.2 – Depths and Elevations of Native Silty Sand

Foundation Unit	Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
North approach	BUG-01	6.1 to 10.8 (borehole termination depth)	352.6 to 347.9	4.7
North abutment	BUG-02	6.6 to 10.0	352.2 to 348.7	3.4
South Approach	BUG-06	4.3 to 6.1	354.5 to 352.8	1.8

SPT ‘N’ values measured in the silty sand ranged from 2 to 17 blows per 0.3 m of penetration, indicating a very loose to compact relative density. The natural moisture contents of samples of the silty sand ranged from 14% to 22%.

Two samples of the silty sand were selected for laboratory grain size analysis testing, the results of which are summarized below. These results are also presented on the Record of Borehole sheets in Appendix A and the grain size distribution curves for these selected samples are plotted on Figure B3 of Appendix B.

Soil Particles	Percentage (%)
Gravel	0 to 3
Sand	50 to 71
Silt	24 to 47
Clay	2 to 3

5.8 Bedrock

The overburden soils described above are underlain by grey metamorphic bedrock of gneissic structure. Occasional horizontal and sub-vertical joints were noted throughout the bedrock cores. The bedrock is generally described as moderately weathered to fresh.

Bedrock was proved by coring in Boreholes BUG-02 and BUG-05 drilled at the north and south abutment, respectively. Table 5.3 summarizes depths and elevations to the top of bedrock or depth to auger refusal in the boreholes.

Table 5.3 – Depths and Elevations of Top of Bedrock / Auger Refusal

Location	Borehole	Top of Bedrock/Auger Refusal	
		Depth (m)	Elevation (m)
North approach	BUG-01	10.8	347.9
North abutment	BUG-02 ⁽¹⁾	13.7	345.0
	BUG-03	13.4	345.3
South Abutment	BUG-04	12.9	345.9
	BUG-05 ⁽¹⁾	14.0	344.8
Norhtwest side of river	BH-1 ⁽²⁾	14.9	-
River	BH-2 ⁽²⁾	12.8	-
Southeast side of river	BH-3 ⁽²⁾	14.5	-

⁽¹⁾ Bedrock proved by coring

⁽²⁾ Boreholes from previous investigation in 1956. (Reference 1)

Total core recovery (TCR) in the bedrock was 100% in all cores. The RQD values typically ranged from 63% to 82%, indicating poor to fair to good rock quality. An RQD value of 0% was recorded for Run 1 in Borehole BUG-05, which was only 0.5 m long and contained a rubble zone. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally 0 to 7. The FI was 15 at the top of Run 2 in Borehole BUG-05.

The estimated unconfined compressive strength of the rock cores generally ranges from 78 MPa to 165 MPa, indicating a strong to very strong rock. A higher estimated unconfined compressive strength of 256 MPa was measured in Borehole BUG-02 Run1, indicating an extremely strong rock. These estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from the boreholes. The results of the point load tests are presented on the Record of Borehole sheet in Appendix A, as average estimated unconfined compressive strength per run. A summary of the Point Load Test Results is presented in Appendix B.

5.9 Water Levels

Water levels were observed in the open boreholes during drilling operations. Two standpipe piezometers were installed at this site, in Boreholes BUG-03 and BUG-04 to monitor water levels after completion of drilling. The water levels measured in the open boreholes and piezometers are summarized in Table 5.4.

Table 5.4 – Water Level Measurements

Location	Borehole	Date	Water Level (m)		Comment
			Depth	Elevation	
North approach	BUG-01	Aug. 7, 2011	2.4	356.3	Open borehole
North abutment	BUG-02	Aug. 7, 2011	3.0	355.7	Open borehole
	BUG-03	Sept. 15, 2011	3.3	355.4	Piezometer
South abutment	BUG-04	Sept. 15, 2011	3.6	355.2	Piezometer
	BUG-05	Aug. 8, 2011	3.0	355.9	Open borehole

The piezometric readings reveal that the groundwater level is at an approximate elevation of 355.3, 3.3 m to 3.6 m below ground surface.

The Preliminary GA drawing indicates that the Bug River water level was at elevation 355.7 m in May 2011.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

6 MISCELLANEOUS

Borehole locations were selected in the field by Thurber Engineering Ltd. Borehole elevations and coordinates were provided by Genivar.

Thurber obtained utility clearances for the borehole locations prior to drilling.

Eastern Ontario Diamond Drilling Ltd. from Hawkesbury, Ontario supplied a truck mounted CME 75 drill rig and conducted the drilling, sampling and in-situ testing operations.

The field program was supervised on a full time basis by Mr. Stephane Loranger of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall planning and supervision of the field program was conducted by Mr. Mark Farrant, P. Eng. Interpretation of the data and preparation of this report were carried out by Ms. Lindsey Blaine, E.I.T. and Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical recommendations for design of a new bridge to replace the existing bridge that carries Highway 105 over the Bug River, approximately 9.8 km south of Highway 125 in the Red Lake District, Ontario.

Highway 105 is a two-lane road. The existing bridge consists of a six-span bridge with a concrete and timber deck. The bridge is supported on timber pile foundation. Each span is approximately 6.0 m long. The total length and width of the bridge are 37.0 m and 9.8 m, respectively. The approach embankments are approximately 3.0 m to 4.0 m high. The bridge was built in 1958 and was rehabilitated in 1989. It is understood the existing timber bridge will be removed and the timber piles will be cut off to the river level.

A single span bridge is being considered for the replacement structure. The bridge designer, Genivar, has indicated that the Region would like to consider supporting the girders on steel sheet pile abutment foundations at this site. Drawings showing the proposed General Arrangement and abutment and piling details are attached in Appendix I. Foundation recommendations are provided for both conventional foundations as well as the steel sheet pile foundation option for the abutments.

The proposed length of the bridge is 30.8 m with a width of 9.5 m. It is anticipated that the replacement structure will be constructed along the existing horizontal alignment. Based on the latest design data, Highway 105 will be raised 720 mm and 500 mm at the south and north abutment, respectively.

The discussions and recommendations presented in this report are based on the factual data obtained during the course of the investigation. The plans used for preparation of this report were provided by Genivar.

8 STRUCTURE FOUNDATIONS

The stratigraphy encountered in the six boreholes drilled at the north and south approaches and abutments revealed pavement structure overlying granular fill. The granular fill was 2.8 m to 3.8 m thick and consisted of loose to dense sand and gravel. Below the fill, layers of native very loose to compact sand and silty sand/sandy silt and firm silty clay were encountered. A layer of organic clayey silt/silty clay was noted in Boreholes BUG-04 and BUG-05 drilled at the south abutment. The thickness of the organic clayey silt/silty clay layer was 0.6 m and 1.4 m. Cobbles and rock fragments were encountered in some boreholes near the borehole termination depths. Grey metamorphic bedrock of gneissic structure was contacted below the overburden at depths ranging from 10.8 m to 14.0 m (elevation 344.8 to 347.9). The bedrock was described as moderately weathered to fresh.

The boreholes for the current investigation were drilled at the existing abutment locations. The latest General Arrangement (GA) drawing shows that the proposed abutments are located about 3.0 m to 4.0 m in front of the existing abutments.

The piezometric readings reveal that the groundwater level is at approximately elevation 355.3, 3.3 m to 3.6 m below ground surface.

The Preliminary GA drawing indicates that the Bug River water level was at elevation 355.7 m in May 2011.

Recommendations are provided for sheet pile foundation walls at the abutments supporting the precast prestressed girders. This is the option that is being considered by the Region.

Recommendations are also provided for the following more conventional foundation types to support the bridge abutments:

- Spread footings on native soils
- Augered Caissons (drilled shafts)
- Driven piles

A comparison of the technical advantages and disadvantages of alternative foundation schemes (sheet pile foundation walls, driven steel H-piles, spread footings on native soil and caissons/drilled shafts) is presented in Appendix E. A foundation scheme preferred from a foundations perspective is recommended.

8.1 Steel Sheet Pile Abutment Walls

As indicated earlier, the Region is considering supporting the girders for the replacement bridge on steel sheet pile abutment foundation. The depth to bedrock in the abutment boreholes varies from 12.9 m to 14.0 m and the sheet pile depth will vary along the alignment of the sheet pile. Sheet piles must be driven to bedrock or to refusal at or below elevations given in Table 8.1.

Table 8.1 – Recommended Sheet Pile Tip Elevation

Foundation Unit	Borehole	Estimated Pile Tip Elevation to Bedrock or Auger Refusal (m)
North Abutment Wall	BUG-02 ⁽¹⁾	345.0
	BUG-03	345.3
South Abutment Wall	BUG-04	345.9
	BUG-05 ⁽¹⁾	344.8

⁽¹⁾Bedrock proved by coring

Based on the borehole data, most of the sheet piles are expected to reach bedrock or a layer of refusal. In the area of Boreholes BUG-02 and BUG-05 cobbles and rock fragments were encountered below elevation 346.0, just above the bedrock/refusal. The boreholes drilled near the existing abutments did not encounter any major obstructions in the sand and gravel embankment fill. It should however be noted that the locations of the proposed abutments are about 3.0 m to 4.0 m in front of the existing abutments and rockfill is visible on the side and forward slopes below the existing abutment. It is not confirmed whether this rockfill is for erosion protection purposes or the embankment contains rockfill. It must be recognized that embankment fills are heterogeneous in nature and may contain obstructions such as boulders or rockfill. If such obstructions are encountered at the proposed location of the new abutments, they will have to be removed to facilitate driving of sheet piles.

The factored Geotechnical Resistance at ULS (per metre width) has been assumed to be 30% of the structural capacity of the sheet pile selected. This reflects the possibility that in light of the variable depth to bedrock or refusal layer along the alignment of the abutment walls, the sheet piles may not rest uniformly on bedrock.

The factored Geotechnical Resistances at ULS (per metre width) recommended for three sheet pile sections driven to bedrock or layer of refusal are as follows:

Table 8.2 – Recommended Axial Resistances of Steel Sheet Piles

Sheet Pile Section	Factored ULS Resistance per meter width (kN)
EZ-88	1,000
XZ-100	1,400
JZ-127	1,850
AZ 36-700 N	1,800
AZ 38-700N	1,900

The SLS condition will not govern for steel sheet piles driven to bedrock.

The abutment reactions and service and ultimate loads were reported by Genivar to be 639 kN/m at ULS and 458 kN/m at SLS.

Steel sheet pile installation should be in accordance with OPSS 903.

Sheet piles should be driven to the specified elevation noted in Table 8.1. The appropriate pile driving note is "Sheet piles to be driven to bedrock or refusal".

The lateral resistance of sheet piles may be computed using the lateral earth pressure distribution and parameters presented in Section 9.

The groundwater levels both in front and behind the sheet pile walls must be considered for the sheet pile design.

Sheet piles should be provided with sheet pile tip protector to minimize any tip damage. Design of the permanent sheet pile walls must consider environmental conditions such as road salts or fluctuating water levels that may cause corrosion and reduce the service life of the structure.

In addition to the option of steel sheet pile abutment wall foundations, the following more conventional foundation options were also considered for the abutments:

8.2 Spread Footings on native soils

Spread footings on native soils are not recommended at this site due to the following reasons:

1. Founding the spread footings on suitably, uniform native soils will require excavation of the fill and layers of organic clayey silt/silty clay and into native permeable, cohesionless sand below the the river level. This excavation will be more than 5.0 m to 6.0 m deep and such an excavation would require extensive dewatering and yet would remain at risk of becoming destabilized due to the inflow of unbalanced groundwater heads.
2. The geotechnical resistance available in the native soils below the fill and organic clayey silt/silty clay is relatively low and there is potential for settlement.
3. Spread footings could be subject to erosion or undermining/scour during high river flows.

In light of the above, the spread footings option was not further developed.

8.3 Augered Caissons (drilled shafts)

Augered caisson foundations were also considered for the support of the bridge. The caissons must be founded on the inferred bedrock at depths in the order of 12.9 m to 14.0 m (elevations 344.8 to 345.9) below original ground surface or roadway embankment.

The base of the caissons would be about 9.8 m to 10.9 m below the groundwater level, resulting in high hydrostatic heads near the base. The caissons will also have to be installed through mainly cohesionless soils under the water table.

Unwatering of the caisson would be impractical and attempts to do so might result in continued flow of fines into the caisson excavation.

Installation of deep caissons to bedrock is also expected to be a more expensive option than driven piles.

For these reasons, the use of a caisson foundation is not recommended.

8.4 Steel H-Pile Foundations

The ground conditions at the site are considered to be suitable for the support of abutments on steel H-pile foundation driven to bedrock.

The anticipated pile tip elevations to reach the bedrock surface or refusal are presented in Table 8.1.

H-piles driven to refusal on bedrock should be designed using the geotechnical resistance presented in Table 8.3.

Table 8.3 – Recommended Pile Resistance Value

Pile Section	Factored Geotechnical Resistance at ULS
HP 310 x 110	2,000 kN

The factored structural resistance of the piles at ULS must be checked by the structural designer as per Section 6.8.8 of the CHBDC. The SLS condition will not govern for piles driven to bedrock.

Oversize materials (e.g. greater than 75 mm nominal diameter) must not be used in any fills through which the piles will be driven.

8.4.1 Pile Tips

The tips of all piles should be fitted with cast steel, H-section pile tip protection from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent.

8.4.2 Pile Installation

Pile installation should be in accordance with OPSS 903. For piles installed for the tolerances shown in Clause 903.07.05.01 of the Specification, the foundation drawing should include the note “Piles to be driven to bedrock”.

For design of similar bridges in the Northwest Region, foundation design has required that the deviation at the top of the piles be limited to 12 mm. To reduce the potential for

misalignment resulting from hard driving to confirm bedrock, it is recommended that the pile driving note on the foundation drawing be modified as follows:

“Piles to be driven to bedrock. Upon initial contact with the bedrock:

1. Apply 10 blows at 10% of the hammer energy. Record the penetration.
2. Apply 10 blows at 50% of the hammer energy. If the penetration under 10 blows is less than 12.5 mm, the pile is set.
3. If the penetration under 10 blows is greater than 12.5 mm, refer the issue to the design team for resolution.”

Use of a driving template or other means may also be required to achieve the specified maximum deviation.

8.4.3 Downdrag

Based on the GA drawing, new fill will be placed behind the sheet pile walls and behind the concrete box girder. The thickness of the new fill will be approximately 2.6 m. At the south abutment, downdrag forces will develop along the length of the pile embedded in the organic clayey silt/silty clay layer. No downdrag loads are anticipated for the piles at the north abutment.

For design purposes, an unfactored downdrag load of 200 kN per pile is recommended to evaluate the impact of downdrag for the piles at the south abutment.

This downdrag load should be multiplied by a load factor of 1.25 as per CHBDC Commentary Clause C8.6.4 to obtain a factored downdrag load.

In accordance with Section 6.8.4 of the CHBDC and clause C6.8.4 of the Commentary to CHBDC, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag.

In geotechnical analysis of downdrag, live load effects should not be considered. The location of the neutral plane for a pile or groups of piles should be determined by using unfactored loads and unfactored geotechnical parameters.

Factored dead and downdrag load should not exceed the factored structural resistance of a pile.

8.4.4 Lateral Resistance

The lateral resistance of a pile in the predominantly cohesionless soils encountered at this site may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h \cdot z / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p \quad (\text{kPa})$$

where

$$z = \text{depth of embedment of pile in metres}$$

$$D = \text{pile width in metres}$$

$$n_h = \text{value from Table 8.4}$$

$$\gamma = \text{unit weight (Table 8.4)}$$

$$K_p = \text{passive earth pressure coefficient (Table 8.4)}$$

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K , for analysis may be obtained by the expression, $K = k_s \cdot L \cdot D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} \cdot L \cdot D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements. It is recommended, however, that the total lateral resistance assumed in one pile be limited to no more than 110 kN at ULS and 40 kN at SLS.

Parameters for lateral pile resistance are shown in Table 8.4. The unit weights provided in the table for soils below the groundwater level are buoyant (effective) unit weights for use in the lateral resistance calculation.

Table 8.4 – Parameters for Lateral Pile Resistance

Location	Elevation	K_p	n_h (kN/m ³)	Unit Weight* (kN/m ³)	S_u (kPa)	Soil Conditions
North abutment wall	OGI to 355.0	3.0	3,000	21	-	Sand and gravel FILL, very loose to dense
	355.0 to 345.0	3.0	2,500	11*	-	Sand, sandy silt/silty sand very loose to compact
South abutment wall	OGI to 355.0	3.0	2,500	21	-	Sand and gravel FILL, very loose to dense
	355.0 to 353.7	3.0	2,000	11*	-	Sand, very loose to loose
	353.7 to 352.5	3.0	-	10*	30	Organic clayey silt/silty clay, silty clay, very soft to firm
	352.5 to 345.0	3.0	2,000	11*	-	Sand, sandy silt/silty sand loose to compact

*Buoyant unit weight below the water table.

Pile interaction should be considered with reference to CHBDC Clause 6.8.9.2.

For lateral soil/pile group interaction analysis, the modulus of subgrade reaction (k_s) may have to be reduced based on pile spacing.

Where a pile group is oriented *perpendicular* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

* D is the width of the pile, and spacing is measured centre to centre

Where a pile group is oriented *parallel* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D	1.00
6 D	0.70
4 D	0.40
3 D	0.25

Intermediate values may be obtained by interpolation.

8.5 Proposed Foundation

From a geotechnical perspective and based on the subsurface conditions steel H-pile foundation driven to bedrock or refusal is the preferred foundation option at this site.

Supporting the bridge girders on the steel sheet pile abutment foundations driven to bedrock is also a feasible option.

8.6 Frost Cover

The depth of frost penetration at this site is 2.5 m. The base of pile caps, if employed, must be provided with a minimum of 2.5 m of earth cover as protection against frost action.

9 EARTH PRESSURE ON SHEET PILE WALLS

Driving of the sheet piles through the existing approach fill and into the underlying very loose to compact native sand, silty sand/sandy silt and silty clay is considered feasible based on the borehole data. However, the latest GA drawing indicates that the proposed abutment locations are approximately 3.0 m to 4.0 m in front of the existing abutments, and no boreholes were drilled at the proposed abutment locations. Also, rockfill is visible on the side and forward slopes below the existing abutment. If such obstructions are present at the location of the sheet pile abutment walls, they will have to be removed in order to drive the sheet piles.

Based on GA drawing dated May 2012, the top of the sheet pile abutment wall will be near elevation 358.2 and the final grade of soil in front of the sheet pile will be near elevation 356.3. Therefore, the sheet pile will be exposed over a height of about 1.9 m between the ground surface and the bridge girder. The river valley in front of the sheet pile walls is at an approximate slope of 2H:1V. The maximum thickness of backfill behind the sheet pile wall is 1.9 m with a further 1.7 m and 1.0 m of fill behind the concrete box girder at the south and north abutments, respectively. Granular A or Granular B Type II backfill should be used behind the sheet pile wall and the box girder.

For this design configuration, earth pressures acting on the sheet pile abutment walls may be assumed to be represented by a triangular distribution governed by the characteristics of the soils being retained by the sheet piles. The pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K(\gamma h + q)$$

Where:

P_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see Table 9.1)

γ = unit weight of retained soil (see Table 9.1)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the sheet pile abutment wall are dependent on the material retained by the wall. Typical values are shown in Table 9.1. Use of at-rest coefficient is recommended in view of the relatively rigid behaviour anticipated for a low abutment wall restrained at the top by the bridge girders.

If any new backfill is required behind the sheet pile walls, it should be in accordance with OPSS 902. Granular backfill should be placed to the extents shown in OPSD 3101.150. All granular material should meet the specifications of OPSS 1010 as amended by Special Provision 110S13. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.

**Table 9.1 – Earth Pressure Coefficient (K)
(Horizontal ground surface for Active Pressure and
Sloping Ground surface for Passive Pressure)**

Condition	Earth Pressure Coefficient (K)							
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$, $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Type III $\phi = 32^\circ$, $\gamma = 21.2 \text{ kN/m}^3$		Existing Sand and Gravel Fill, Native Sand and Silty Sand/Sandy silt $\phi = 30^\circ$, $\gamma = 20 \text{ kN/m}^3$		Silty Clay $\phi = 28^\circ$ $\gamma = 18 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind or in front of the Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind or in front of the Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind or in front of the Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind or in front of the Wall (2H:1V)
Active (Unrestrained Wall) K_a	0.27	0.38	0.30	0.46	0.33	0.53	0.36	0.65
At rest (Restrained Wall) K_o	0.43	0.43	0.47	0.47	0.5	0.50	0.53	0.53
Passive (Movement Towards Soil Mass) K_p	3.7	2.1	3.3	1.7	3.0	1.5	2.8	1.2

The factors in Table 9.1 are ultimate values and require certain movements for the respective condition to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to CHBDC.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II.

The design of the sheet pile abutment walls must incorporate measures such as subdrains to permit drainage of the sheet pile abutment wall backfill, or alternatively the sheet pile abutment walls should be designed to withstand the potential build-up of hydrostatic pressures behind the walls.

10 EXCAVATION AND GROUNDWATER CONTROL

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the sand and gravel fill forming the existing approach

embankment and the underlying native sand/sandy silt may be classified as Type 3 soil above the water table and Type 4 below the water table.

The excavation and backfilling must be carried out in accordance with OPSS 902.

The selection of the method of excavation is the responsibility of the contractor and must be based on his equipment, experience and interpretation of the site conditions. Excavations should be inspected regularly for evidence of instability.

GA drawing indicates that water level of Bug River at the bridge location was at elevation 355.7 m in May 2011.

Based on the proposed bridge design, excavation below the groundwater level to construct the new sheet pile abutment wall is not anticipated.

However, the Contractor should be prepared to pump from sumps to remove any seepage water or surface water collecting in an excavation.

Excavation below the groundwater level without prior dewatering is not recommended since the inflow of groundwater will cause boiling and sloughing of the soil below the water table making it difficult to maintain a dry, sound base on which to work.

11 APPROACH EMBANKMENTS

Based on a contour drawing provided by Genivar, it was estimated that the existing approach embankment is up to 3.0 m to 4.0 m high. The foundation soils governing stability of the approach embankments consist generally of existing native very loose to compact sand and silty sand/sandy silt layers.

Communication with Genivar indicates that the existing Highway 105 grade will be raise 720 mm and 500 mm at the south and north abutments, respectively. However, additional fill will be placed behind the new sheet pile walls and the box girder. This new fill is expected to have a maximum thickness of about 3.3 m at the south abutment and 2.9 m at the north abutment. The sides of the new approach fill will be contained by sheet pile walls installed along each edge of the road.

Embankment construction and widening should be carried out in accordance with OPSS 206.

Comments regarding stability of approach embankment slopes and settlement of the foundations soils are provided in the following sections.

11.1 Slope stability

The existing embankments bearing on the foundation soils at this site appear to be performing satisfactorily under the existing conditions. Highway 105 will be raised 720 mm and 500 mm at the south and north abutment, respectively.

The additional approach fill to be placed behind the new abutment will be supported within a sheet pile enclosure and therefore the stability of the new approach will be governed by

the sheet pile wall design. A global slope stability analysis was conducted to assess the embedment requirements for a sheet pile supporting the new approach. The analyses were carried out using the Morgenstern-Price method of slope stability analysis.

The results of the analysis indicate that for fill placed within a sheet pile enclosure driven to bedrock, stability is not an issue.

However, for the alternative design of an H-pile foundation with a sheet pile enclosure behind the H-piles, the results of the analyses indicate that an adequate factor of safety for the long term conditions of 1.5 is achieved if the sheet pile is driven to elevations 353.4 and 352.6 at the north and south abutments, respectively. The factor of safety for the seismic analysis is 1.4, which is considered acceptable for cohesionless soils.

The slope stability computation outputs are included in Appendix F.

11.2 Settlement

The placement of approximately 3.3 m and 2.9 m of new fill behind the sheet pile abutments, including the new fill to raise the Highway 105 grade, will induce immediate (elastic) settlement in the existing cohesionless fill and sand layers. At the south abutment, time dependent (consolidation) settlement in the underlying organic clayey silt/silty clay layer will also be induced by the placement of new fill.

The total immediate and consolidation settlements were assessed using elastic methods and one-dimensional consolidation theory. Based on these analyses, the settlement at the south abutment will be in the order of 35 mm to 40 mm and at the north abutment will be about 20 mm to 25 mm.

The settlement at the north abutment will be immediate in nature and is anticipated to be completed by the end of construction. The consolidation settlement at the south abutment is expected to be completed within 2 months of completion of fill placement. Inspection of the roadway surface and padding of the asphalt at the approaches to re-establish grades as necessary should be implemented during and after construction.

12 EROSION PROTECTION

Erosion protection should be provided along the toe of any slopes that may be in contact with the river flow.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS 804.

13 ROADWAY PROTECTION

During the new bridge construction, temporary excavation of existing embankments will be required. The bridge construction will be done in stages in order to keep at least one highway lane

operational. Roadway protection will be required to facilitate staging of removals and support the existing Highway 105 adjacent to the excavation.

Roadway protection should be provided in accordance with OPSS 539 and designed for Performance Level 2.

Conventional steel soldier pile and timber lagging walls is an available option to provide temporary support to the roadway during excavation. Timber lagging boards should be installed as soon as the soil face is exposed and properly prepared.

The following parameters apply for design of the temporary shoring system:

γ	=	21 kN/m ³	(bulk unit weight)
γ_w	=	11 kN/m ³	(submerged unit weight under groundwater table)
K_a	=	0.33	(Active pressure coefficient for road embankment fill)
	=	0.33	(Active pressure coefficient for native sand/silt)
K_p	=	3.0	(Passive pressure coefficient for road embankment fill)
	=	3.0	(Passive pressure coefficient for native sand/silt)
h_w	=	0	(assuming that the groundwater is maintained below the base of the excavation and that there is no hydrostatic pressure build-up behind a presumably permeable wall)
h_w	=	355.7	(elevation for hydrostatic pressure build-up behind wall)

The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system.

Temporary groundwater and surface water control measures may be required during construction.

The design of roadway protection is the responsibility of the Contractor. All shoring systems should be designed by a Professional Engineer experienced in such designs.

14 SEISMIC CONSIDERATIONS

14.1 Seismic Design Parameters

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.0
- Acceleration Related Seismic Zone 0

- Zonal Acceleration Ratio 0.0
- Peak Horizontal Acceleration 0.02

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.0 should be used in seismic design.

14.2 Liquefaction Potential

The site overlies very loose to dense cohesionless deposits and a high water table is present at this site.

Localized liquefaction during a seismic event may result in local toe failure or minor embankment settlement, but this is expected to be readily repairable.

14.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. The

coefficients of horizontal earth pressure for seismic loading presented in Table 14.1 may be used:

Table 14.1 – Earth Pressure Coefficients for Earthquake Loading

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I or Type III $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	Existing Sand and Gravel Fill, Native Sand/Silt $\phi = 30^\circ$ $\gamma = 20 \text{ kN/m}^3$	Silty Clay $\phi = 28^\circ$ $\gamma = 18 \text{ kN/m}^3$
Active (K_{AE})*	0.28	0.32	0.34	0.37
Passive (K_{PE})	3.3	2.8	2.6	2.4
At Rest (K_{OE})**	0.45	0.50	0.52	0.56

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods

15 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- Based on the borehole data at the locations drilled, driving sheet piles near the location of the existing abutment is feasible. However, the new abutments are located approximately 3.0 m to 4.0 m in front of the existing abutments, where no boreholes were drilled. Visual site inspection indicates presence of rockfill on the side and forward slopes below the existing abutments. It is not confirmed whether this rockfill is for erosion protection or whether the embankment contains rockfill. It must be recognized that embankment fills are heterogeneous in nature and may contain obstructions such as boulders or rockfill. If the sheet piles encounter these conditions, the obstructions will have to be removed to facilitate driving of sheet piles.
- The depth to bedrock has been shown in the investigation to be variable. Since the elevation of the bedrock surface was only established at discrete points, it is possible that higher or lower bedrock elevations will be encountered during construction.
- Roadway protection must be provided to maintain traffic during construction. Temporary shoring systems should be properly designed by a Professional Engineer experienced in such designs.
- The embankment side slopes should be inspected after construction for surficial disturbance. Where necessary, erosion control measures must be implemented.

16 CLOSURE

Engineering analysis and preparation of the report were carried out by Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundations Projects.

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Appendix A

Record of Borehole Sheets (current investigation)

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$




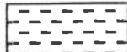
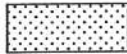


Water Level

C_{pen}

Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION		SYMBOLS	
Fresh (FR)	No visible signs of weathering.		
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
		Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
		Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS	
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

RECORD OF BOREHOLE No BUG-01

1 OF 2

METRIC

W.P. 6942-10-00 LOCATION N 281 06.6 E 358.7 Bug River Bridge ORIGINATED BY SLL
 HWY 105 BOREHOLE TYPE Hollow Stern Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.08.07 - 2011.08.07 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			
358.7 0.0 0.1	ASPHALT: (125mm) SAND and GRAVEL, trace silt and clay Compact to Very Loose Brown Moist (FILL)						20 40 60 80 100	20 40 60 80 100	20 40 60	GR SA SI CL	
			1	SS	23		○ UNCONFINED + FIELD VANE				
			2	SS	3		● QUICK TRIAXIAL × LAB VANE				
			3	SS	13						
355.7 3.0	Silty CLAY, trace sand, occasional rootlets Firm Brown		4	SS	7						
	Grey		5	SS	6						
352.6											
6.1	Silty SAND, trace clay, trace gravel Loose Grey Wet		6	SS	4						
			7	SS	7						
			8	SS	5						
			</								

Continued Next Page

+ ³ × ³ : Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BUG-01

2 OF 2

METRIC

W.P. 6942-10-00 LOCATION N 281 06.6 E 358.7 Bug River Bridge ORIGINATED BY SLL
HWY 105 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2011.08.07 - 2011.08.07 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
	Continued From Previous Page												
347.9	Silty SAND, trace clay, trace gravel Loose Grey Wet		9	SS	100/		348						
10.8	END OF BOREHOLE AT 10.8m UPON REFUSAL ON PROBABLE BEDROCK OR BOULDER. WATER OBSERVED AT 2.4m DURING DRILLING. BOREHOLE GROUTED WITH BENTONITE HOLEPLUG TO 1.9m, SAND TO 0.2m, THEN ASPHALT TO SURFACE.				0.150								

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No BUG-02

2 OF 2

METRIC

W.P. 6942-10-00 LOCATION N 280 93.1 E 973.3 Bug River Bridge ORIGINATED BY SLL
 HWY 105 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.08.07 - 2011.08.07 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
	Continued From Previous Page							SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL × LAB VANE						
								WATER CONTENT (%)						
								20	40	60	80	100		
								PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT				
								W _p	W	W _L				
10.0	SAND, trace gravel, trace to some silt and clay Loose Grey Wet		9	SS	4		348							
			10	SS	7		347							1 89 10 (SI+CL)
							346							
	Compact Some silt, occasional cobbles or bedrock fragments		11	SS	21									
345.0													FI	
13.7	BEDROCK, moderately weathered, grey with white bands Coring started at 13.7m Sub-vertical joints at 13.8m, 14.1m, 14.7m, 14.8m, 15.2m, 15.3m, 15.8m and 16.1m		1	RUN			345						3	RUN #1 TCR=100% SCR=100% RQD=82% UCS=152MPa (Average)
													3	
													0	
							344						2	
													1	
													4	
													0	
							343						1	
													2	
342.3														
16.4	END OF BOREHOLE AT 16.4m. WATER OBSERVED AT 3.0m DEPTH DURING DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 1.8m, AUGER CUTTINGS TO 0.6m, SAND TO 0.3m, THEN ASPHALT TO SURFACE.													

+³ . ×³ : Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BUG-03

1 OF 2

METRIC

W.P. 6942-10-00 LOCATION N 281 02.0 E 972.3 Bug River Bridge ORIGINATED BY SLL
 HWY 105 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.08.08 - 2011.08.08 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
358.7												
0.0	ASPHALT: (150mm)											
0.2	SAND and GRAVEL, trace silt and clay Dense to Compact Brown Moist (FILL) Occasional cobbles		1	SS	49		358					
			2	SS	19		357					
			3	SS	19		356					36 59 5 (SI+CL)
			4	SS	8		355					
355.0	Loose Wet											
3.7	SAND, trace gravel Loose Grey Wet		5	SS	5		354					
353.2	Sandy SILT, trace clay Loose to Very Loose Grey Wet		6	SS	6		353					0 22 71 7
5.5							352					
			7	SS	2		351					
350.2	SAND, some silt to silty, trace clay Compact Grey Wet		8	SS	11		350					
8.5							349					

Continued Next Page



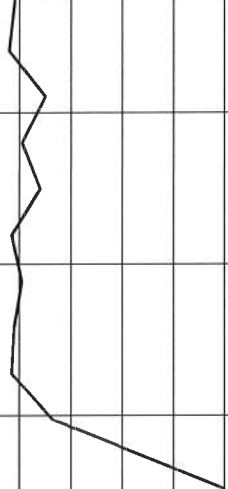
+ 3 × 3 : Numbers refer to
Sensitivity 20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BUG-03

2 OF 2

METRIC

W.P. 6942-10-00 LOCATION N 281 02.0 E 972.3 Bug River Bridge ORIGINATED BY SLL
 HWY 105 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.08.08 - 2011.08.08 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE									
	Continued From Previous Page							● QUICK TRIAXIAL	× LAB VANE									
	SAND, some silt, trace clay Loose Grey Wet		9	SS	5		348											
								347										
					10	SS	5		346									
345.3																		
13.4	END OF BOREHOLE AT 13.4m UPON AUGER REFUSAL ON PROBABLE BEDROCK OR BOULDER. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Sep.15/11 3.3 355.4																	

RECORD OF BOREHOLE No BUG-04

1 OF 2

METRIC

W.P. 6974-10-00 LOCATION N 280 62.1 E 100 4.2 Bug River Bridge ORIGINATED BY SLL
 HWY 105 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.08.09 - 2011.08.09 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _P	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE								
358.8																			
0.0	ASPHALT: (150mm)																		
0.2	SAND and GRAVEL, trace silt and clay Compact to Loose Brown Moist (FILL)		1	SS	30														
			2	SS	4														
			3	SS	4														
			4	SS	12														
354.9																			
4.0	Silty CLAY Firm Grey		5	SS	5														
354.0	Layer of organic clayey SILT (600mm) Dark Brown to Black Wet																		
4.9																			
353.4																			
5.5	SAND, trace silt and clay, occasional cobbles Loose Brown Wet		6	SS	9														
			7	SS	4														
			8	SS	1														
	Very Loose																		

Continued Next Page

+³ ×³: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BUG-04

2 OF 2

METRIC

W.P. 6974-10-00 LOCATION N 280 62.1 E 100 4.2 Bug River Bridge ORIGINATED BY SLL
 HWY 105 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.08.09 - 2011.08.09 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
	Continued From Previous Page													
	SAND, trace to some silt and clay Loose Grey Wet		9	SS	6		348							
							347							
	Compact Some gravel to gravelly		10	SS	31									26 62 12 (SI+CL)
345.9														
12.9	END OF BOREHOLE AT 12.9m UPON AUGER REFUSAL ON PROBABLE BEDROCK OR BOULDER. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Sep.15/11 3.6 355.2													

+³, X³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BUG-05

1 OF 2

METRIC

W.P. 6974-10-00 LOCATION N 280 59.0 E 100 0.3 Bug River Bridge ORIGINATED BY SLL
 HWY 105 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.08.08 - 2011.08.09 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								20 40 60 80 100					
358.9													
0.0	ASPHALT: (150mm)												
0.2	SAND and GRAVEL, trace silt and clay Dense Brown Moist (FILL)		1	SS	46		358						
	Compact		2	SS	11		357						36 57 7 (SI+CL)
			3	SS	10								
355.9							356						
3.0	SAND, trace gravel, occasional wood fragments Loose to Very Loose Brown Wet		4	SS	9		355						
			5	SS	4		354						
353.7													
5.2	Layer of organic silty CLAY/clayey SILT, with wood fragments, trace rootlets Very Soft Dark Brown Wet		6	SS	0		353						
352.3							352						
6.6	SAND, some gravel, trace silt and clay Loose Brown Wet		7	SS	7		351						11 85 4 (SI+CL)
			8	SS	5		350						
							349						

Continued Next Page

+ 3 . X 3 : Numbers refer to
Sensitivity 15 5 10 (%) STRAIN AT FAILURE

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No BUG-06

1 OF 2

METRIC

W.P. 6942-10-00 LOCATION N 280 57.3 E 100 8.0 Bug River Bridge ORIGINATED BY SLL
 HWY 105 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.08.08 - 2011.08.08 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
358.9													
0.0	ASPHALT: (125mm)												
0.1	SAND and GRAVEL, trace silt and clay Compact Brown Moist		1	SS	28		358						
			2	SS	11		357						
	Dense		3	SS	42								
356.0													
2.9	Silty CLAY, trace sand Firm Grey		4	SS	6		356						0 5 52 43
							355						
354.5													
4.3	Silty SAND, trace clay Compact Brown Moist to Wet		5	SS	17		354						
352.8							353						
6.1	SAND, trace to some silt and clay, trace gravel Very Loose to Compact Grey Wet		6	SS	3								
							352						
			7	SS	7		351						3 87 10 (SI+CL)
			8	SS	10		350						
							349						

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BUG-06

2 OF 2

METRIC

W.P. 6942-10-00 LOCATION N 280 57.3 E 100 8.0 Bug River Bridge ORIGINATED BY SLL
 HWY 105 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.08.08 - 2011.08.08 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
	Continued From Previous Page							20	40	60	80	100				
347.6	SAND, trace to some silt and clay, trace gravel Compact Grey Wet		9	SS	25		348									
11.3	END OF BOREHOLE AT 11.3m. BOREHOLE BACKFILLED WITH HOLEPLUG BENTONITE TO 1.5m, CUTTINGS TO 0.1m, THEN CONCRETE TO SURFACE.															

+³, X³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

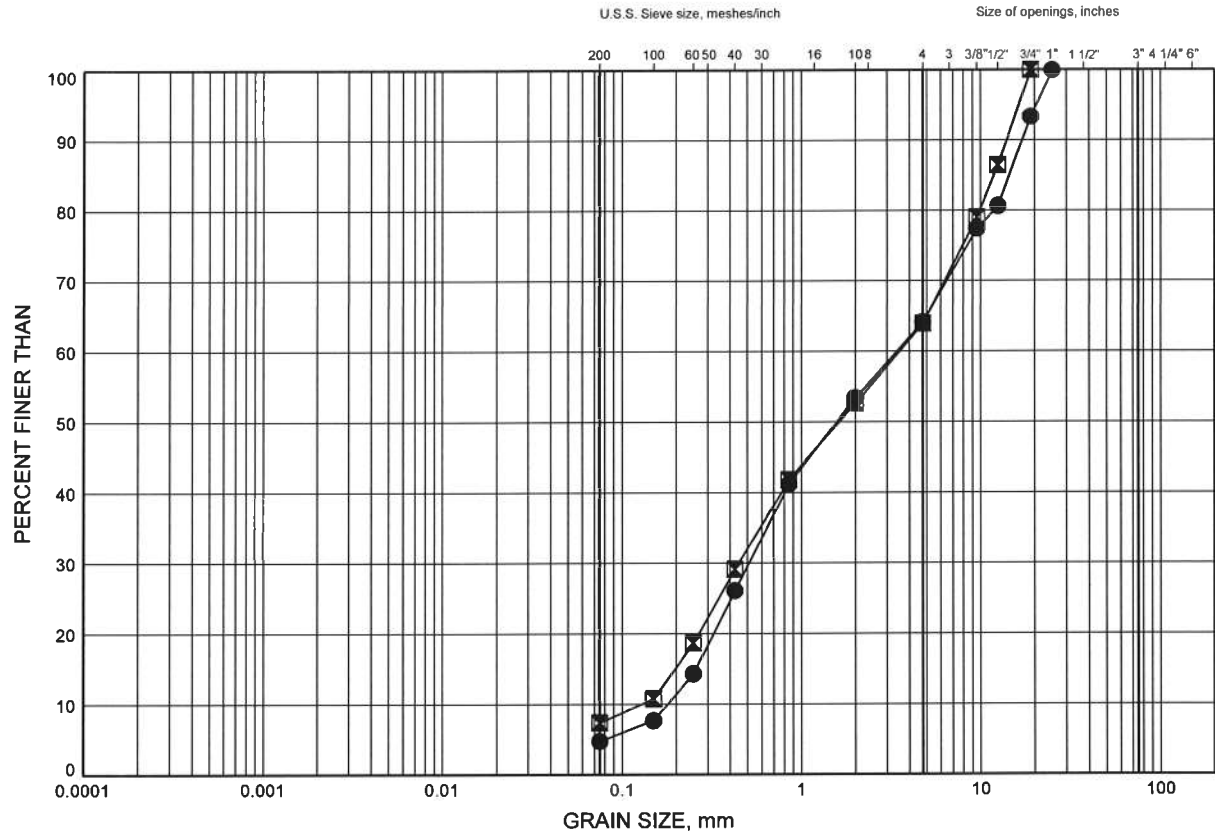
Appendix B

Laboratory Test Results

NWR HWY 11 Bridge GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND & GRAVEL FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

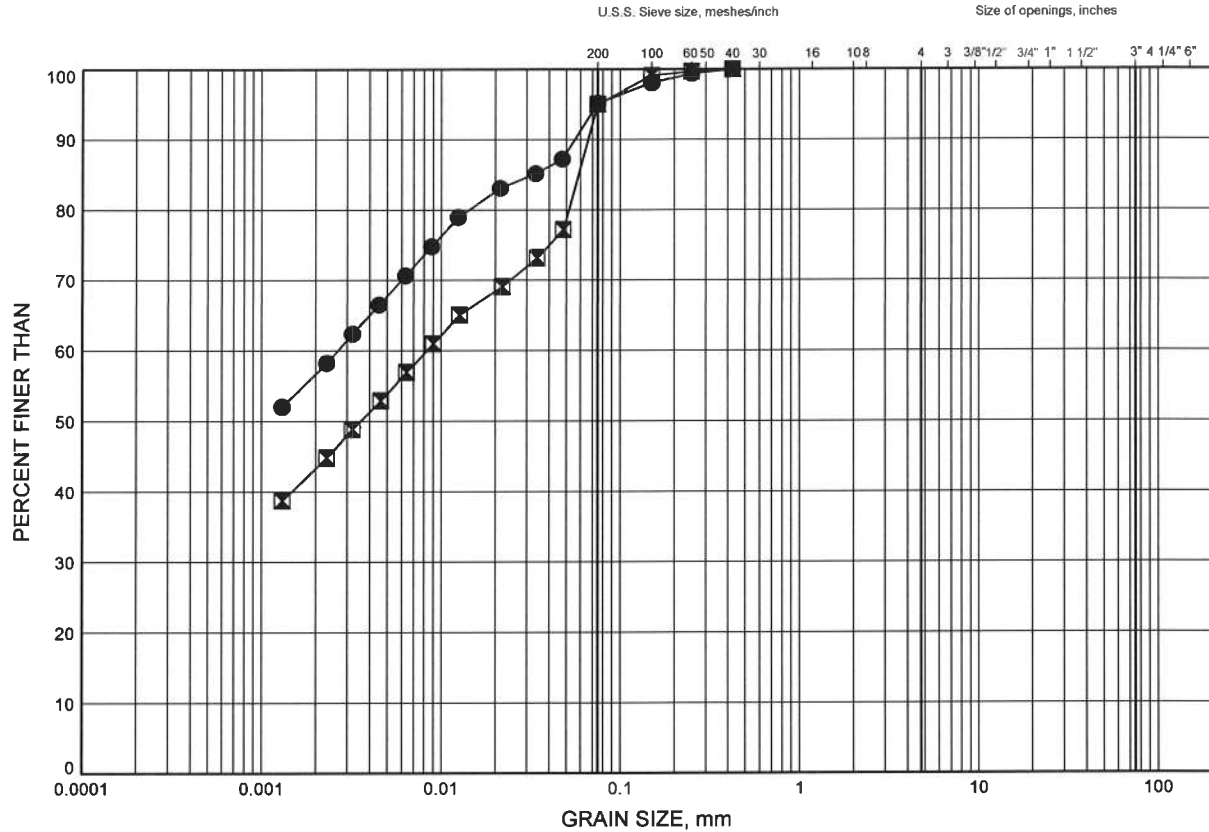
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BUG-03	2.59	356.09
■	BUG-05	1.83	357.03

NWR HWY 11 Bridge GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY CLAY



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

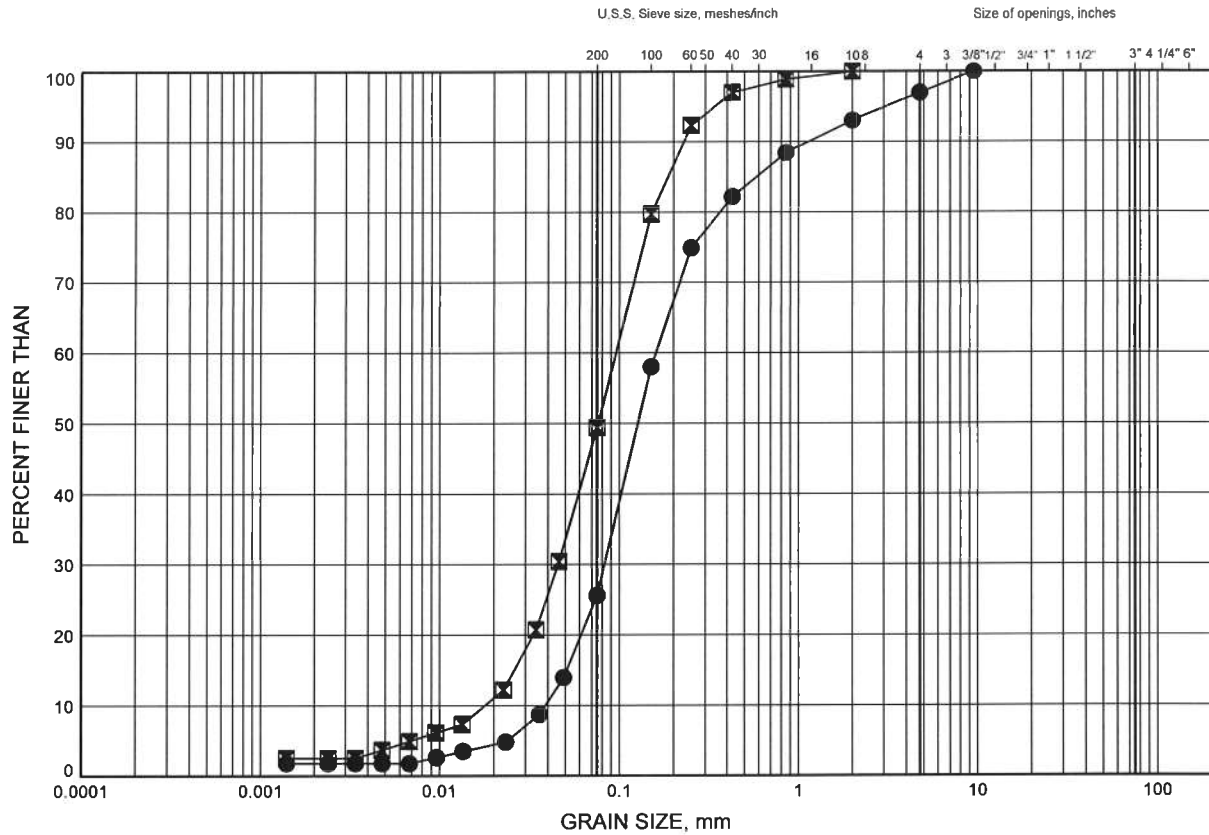
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BUG-01	3.35	355.34
■	BUG-06	3.35	355.53

NWR HWY 11 Bridge GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY SAND



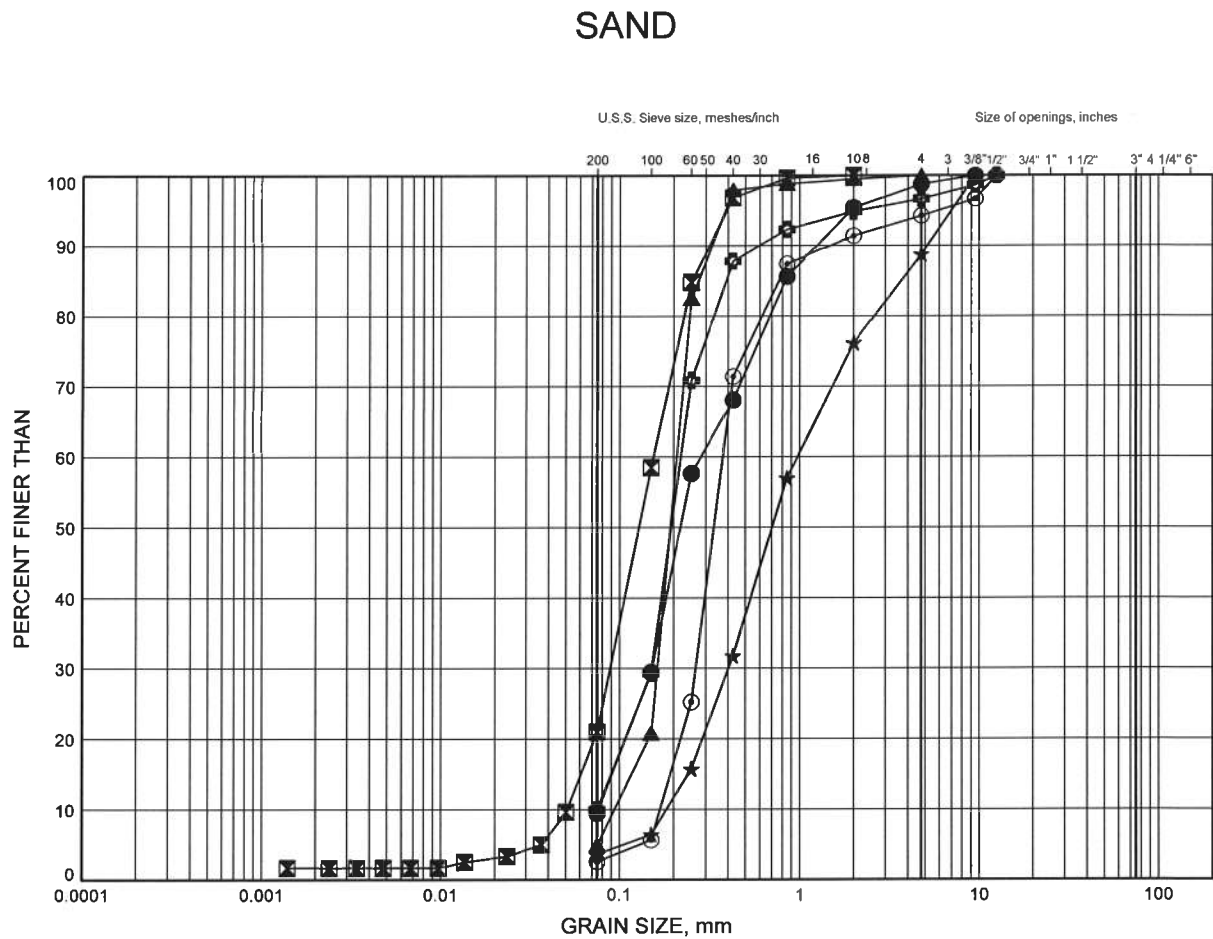
SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BUG-01	7.92	350.77
■	BUG-02	8.84	349.87

NWR HWY 11 Bridge GRAIN SIZE DISTRIBUTION

FIGURE B4



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

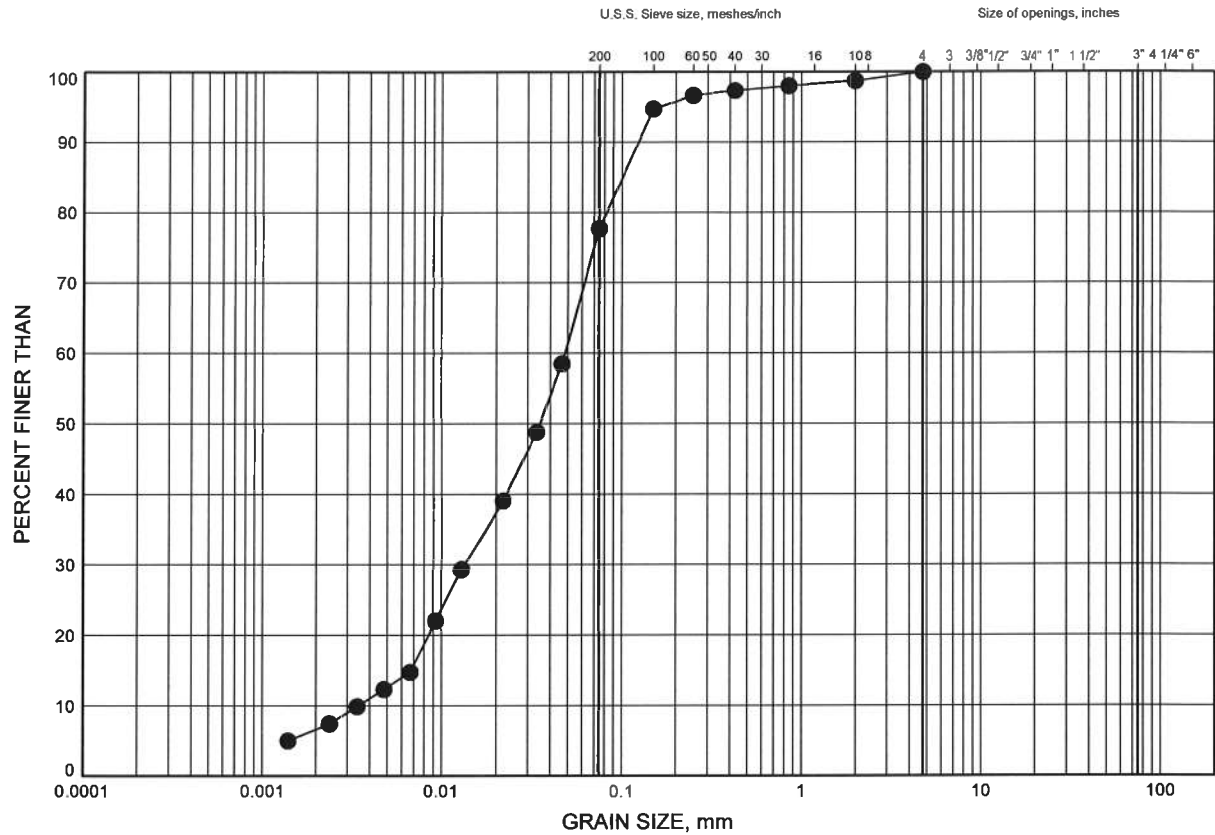
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BUG-02	11.89	346.82
⊠	BUG-03	10.97	347.71
▲	BUG-04	7.92	350.92
★	BUG-05	7.32	351.54
⊙	BUG-05	10.36	348.49
⊕	BUG-06	7.92	350.96

NWR HWY 11 Bridge GRAIN SIZE DISTRIBUTION

FIGURE B5

SANDY SILT



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BUG-03	6.40	352.28

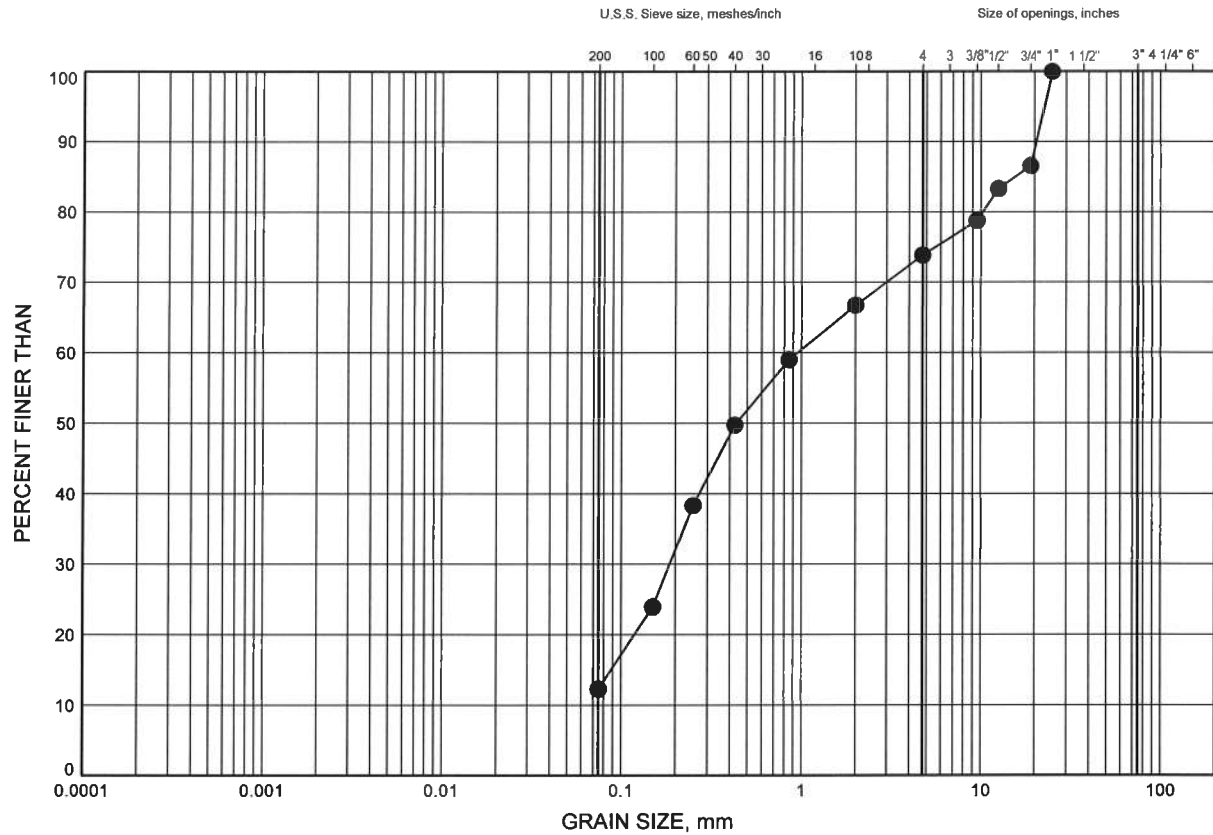


W.P.# 6942-10-00
Prepared By AN
Checked By RPR

NWR HWY 11 Bridge GRAIN SIZE DISTRIBUTION

FIGURE B6

Some GRAVEL to GRAVELLY SAND



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

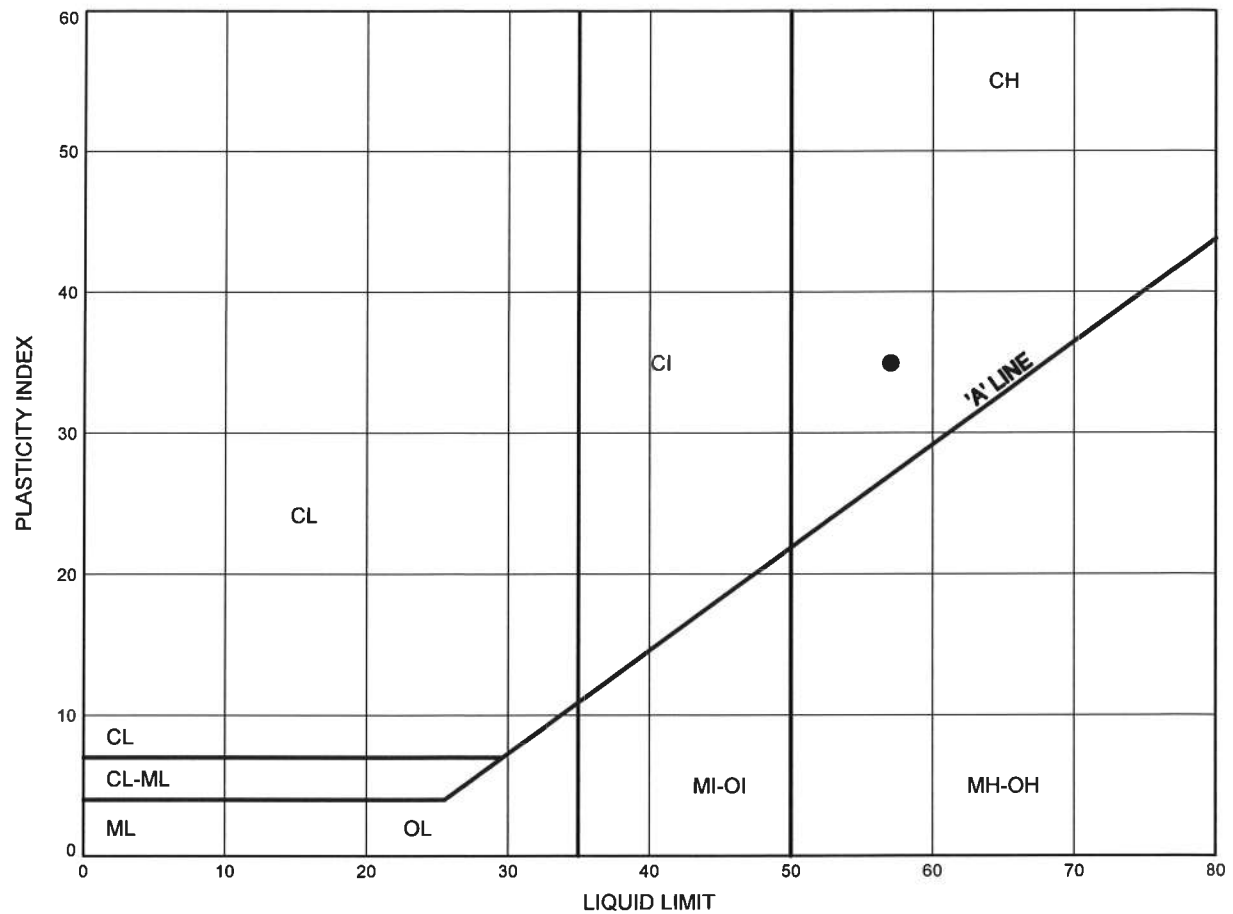
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BUG-04	12.50	346.35

NWR HWY 11 Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE B7

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BUG-01	3.35	355.34

Date January 2012
 Project 6942-10-00



Prep'd AN
 Chkd. RPR



THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

POINT LOAD TEST SHEET

Job No : 19-5308-40 Client : GENIVAR
Date Drilled : August 07/ 2011
Project Name : Bug River Bridge Date Tested : September 06/ 2011
Core Size : NQ BH No : BUG-02 Tester : DB

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	13.8	D	15.4	46.7	78.5	164.5	Metamorphic	Very Strong
2	1	14.4	D	11.6	47.4	61.0	120.4	Metamorphic	Very Strong
3	1	15.0	A	15.7	47.4	47.2	136.5	Metamorphic	Very Strong
4	1	15.6	A	11.0	47.4	56.1	83.2	Metamorphic	Strong
5	1	16.1	D	24.6	47.5	54.0	255.7	Metamorphic	Extremely Strong
6									
7									
8									
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

POINT LOAD TEST SHEET

Job No : 19-5308-40 Client : GENIVAR
Date Drilled : August 09/ 2011
Project Name : Bug River Bridge Date Tested : September 06/ 2011
Core Size : NQ BH No : BUG-05 Tester : DB

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	2	14.7	D	10.6	47.5	61.9	110.1	Metamorphic	Very Strong
2	2	15.2	A	11.1	47.6	61.1	78.3	Metamorphic	Strong
3	2	17.4	A	23.4	48.3	61.0	164.2	Metamorphic	Very Strong
4									
5									
6									
7									
8									
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									

- * It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
* Diametral Test should have $0.7 \times D$ on either side of test point.

Appendix C

Report from previous investigation (1956)

56-F-2200

Hwy. 105

BUG RIVER

B.A.532

RACEY, MACCALLUM AND ASSOCIATES
LIMITED

A COMPANY OWNED, DIRECTED AND OPERATED BY

Consulting Engineers
AND ASSOCIATED STAFF

MONTREAL  VANCOUVER

TORONTO

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TORONTO DIVISION
20 CARLTON STREET

REPORT NO. D-500/T-262

June 5th, 1956.

A.N. Teye, Esq.,
Bridge Engineer,
Department of Highways of Ontario,
Bridge Office, 13th Floor,
East Block, Parliament Building,
Queen's Park,
Toronto, Ontario.

Attention: Mr. S. McCombie.

56-F-220C

RE: REPORT ON A FOUNDATION INVESTIGATION
FOR THE BUG RIVER BRIDGE, HIGHWAY
NO. 105, RED LAKE DISTRICT, ONTARIO.

Dear Sirs:

We have completed the foundation investigation at the Bug River Bridge Site in the Red Lake District and prepared the attached report, with details on the presently existing timber trestle bridge and recommendations for the foundation of the new bridge.

For your convenience, we wish to summarize here that one foot diameter piles for the proposed pile foundation are estimated to have a safe bearing capacity between 20 and 26 tons each, at 25 to 35 foot depth below the bridge deck, depending on the pile locations (shown on enclosure h, below). With regard to settlement, an allowable load for piles at 3 foot spacing, of about 15 tons per pile would apply if only 3/4 inch differential settlement is tolerated. By applying the higher safe loads settlement would be proportionately higher.

We trust that the information contained in the report will be of help in the bridge design.

Yours very truly,

RACEY, MACCALLUM AND ASSOCIATES LIMITED.

K. Tubbs

E. TUBBSING, P.Eng.

KT/EM

Original & 3 copies - Department of Highways of Ontario, Toronto.

Att: Mr. S. McCombie.

- 1 - R.M.A. Montreal.
- 2 - Soils Engineers.

REPORT ON A FOUNDATION INVESTIGATION
FOR THE BBO RIVER BRIDGE, HIGHWAY NO.105,
RED LANS DISTRICT, ONTARIO.

Report No: E-500/T-262

Racey, MacCallum & Associates Ltd.

June 6th, 1956.

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THE FIELD WORK	1
THE SOIL CONDITIONS	2
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ENCLOSURES

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TOPOGRAPHIC SKETCH SHOWING THE LOCATION OF THE SITE.	1 (above)
LAYOUT SKETCH SHOWING THE LOCATION OF THE BOREHOLES AND OTHER PERTINENT DATA.	1 (below)
PHOTOGRAPHS. FIGS. 1, 2 & 3	2
LONGITUDINAL SECTIONAL SOIL PROFILE	3
TABLE GIVING AVERAGE PENETRATION RESISTANCE	4 (above)
SECTIONAL PROFILE WITH SAFE BEARING VALUES AT VARIOUS POINT ELEVATIONS	4 (below)
ENGINEERING DATA SHEETS	5, 6 & 7

June 5th, 1956.

**REPORT ON A FOUNDATION INVESTIGATION
FOR THE BUG RIVER BRIDGE, HIGHWAY NO. 105.
RED LAKE DISTRICT, ONTARIO.**

This report covers a foundation investigation for the proposed new highway No. 105 bridge crossing the Bug River in the Red Lake District, Ontario.

THE LOCATION OF THE SITE:

This bridge site is located on highway No. 105 approximately two miles South East of the village of Red Lake, where the highway crosses the Bug River which connects Gallrock Lake with Stone Lake.

THE PRESENTLY EXISTING BRIDGE:

Prior to the field work the bridge was visited in March 1956, and three photographs were taken in order to assist thereby somehow in the design of the new bridge.

The present timber trestle bridge is 120 feet long, 20 feet wide and has bents at 15 foot spacing. The bridge deck was 10 feet above the ice cover of the river. The river embankments slope gradually on either side. The approaches consist of pit run gravel fill. The photographs (figures 1 and 2, encl. 2) show some more details. Figure 3 shows how the bridge was jacked into the required elevation by inserting timbers, 15 feet from the end of the bridge.

THE LOCATION OF THE BOREHOLES:

The location of the three boreholes carried out is as shown on the layout sketch (encl. 1, below) one borehole being drilled near either end of the bridge and the third one from the centre of the bridge deck.

THE FIELD WORK:

The drilling equipment was brought to the site on April 9th, 1956, and drilling began immediately on borehole 1, which was completed on April 11th. Borehole 2 was carried out from April 12th to 13th and borehole 3 was started on April 14th and completed on April 16th. The equipment was subsequently loaded and removed from the site.

Drilling was performed with a standard diamond core drill under moderate weather conditions. Sampling was carried out with standard two inch diameter split tube samplers or with Shelby tube samplers, whatever promised to retain the soil sample best.

The penetration of the sampler was recorded for both types of samplers though the correlation to soil density has been established

THE FIELD WORK: (Cont'd).

roughly only for the two inch diameter split tube sampler.

Continuous penetration resistance can be obtained by driving the two inch diameter conical drive point, supplementary to the general drilling, and nearby. This was done for all three boreholes and a correlation of 75% cone resistance approximately equal to the standard penetration resistance value considered permissible.

In borehole 1 the cone penetration test had to be repeated since it was found that the first cone, driven after the completion of the borehole, found its way into the borehole, thereby indicating lower resistance than was to be expected.

THE SUBSOIL CONDITIONS:

A longitudinal sectional soil profile is presented on enclosure 3.

Under the 12 to 13 feet of pit run gravel fill of the abutments the embankments show a varying sequence of fine to medium sand, silty sand and, particularly in borehole 2, embedded organic matter, possibly flat pockets of very limited size only. In the centre of the river very fine and fine sand occupies the space from the river bottom to about 17 feet below, the deeper material consisting of coarser material, however, no gravel. An increase in grain size and density is found in all holes at the approximate depth of 35 to 40 feet below the bridge deck. Some gravel was noticed in these low beds.

The density of the sand is best characterized by the penetration curves of the two inch conical drive point plotted on the attached Engineering Data Sheets (enclosure 5 to 7). The equivalent penetration of the drive point has been averaged for five foot intervals in a diagram (enclosure 4, above). This diagram will be referred to below for tentative computations of bearing capacities. The scatter of the penetration resistance with depth is comparatively small.

Bedrock, consisting of a very hard dark gray, metamorphic, basic rock and siliceous rock of gneissic structure, occurs in the E.W. below hole 1 at 49 ft. depth, rising gradually to 42 ft. depth in the S.E. below hole 2. Refusal met in hole 3 is in good accordance with this dip of the rock surface.

DISCUSSION OF THE RESULTS:

The sand as a bearing soil is judged from its density. The equivalent number of blows representing the penetration resistance of the sand increases with depth with the exception of a looser zone within 30 to 35 ft. depth (from the bridge deck) below the river bed.

For the determination of approximate bearing capacities the lowest penetration resistance recorded is utilized.

The type of foundation considered here is limited to a

DISCUSSION OF RESULTS: (Cont'd).

pile foundation with 1 foot diameter piles arranged in rows for the bents of the future bridge as it is understood that this is the required pier type.

The bearing capacity of piles has been investigated by G.G. Meyerhof * theoretically, and in a very recent publication ** the immediate utilization of the standard penetration resistance for the determination of approximate bearing capacities has been suggested by the same author.

The ultimate bearing capacity is accordingly determined from:

$$Q_f = q N A_p + \frac{\bar{N} A_s}{50} \quad (\text{tons})$$

where \bar{N} = average penetration resistance (blows/ft) near pile point,

A_p = sectional area (sq.ft.) of pile point,

\bar{N} = average penetration resistance within embedded length of pile, and

A_s = surface area (sq.ft.) of the embedded length of pile.

This formula applies to saturated sand, no reduction is, therefore, needed for submerged condition.

Safe bearing values, with a factor of safety of three, have been estimated on the basis of the above and the results are entered in a diagram and in a sectional sketch (enclosure h, above right and below). The latter showing safe bearing values for some selected pile locations and point elevations.

With a factor of safety of three a preferable load of 20 tons per pile of 1 foot diameter would necessitate driving the piles to about El. 75 through the present approach fill or to about El. 65 in the centre of the river (El. 100 being the elevation of the bridge deck). The greater depth in the river is mainly due to the lower density of the sand in the river above El. 65.

The given values for safe bearing capacity of single piles are based on the lowest penetration resistances recorded and, therefore,

*) G.G. Meyerhof, The Ult. Bearing Cap. of Found. Geotechnique 1951.

**) G.G. Meyerhof, Penetration Tests and Bearing Capacity of Cohesionless Soils. J.ASCE 1956.

DISCUSSION OF RESULTS (Cont'd).

the actual capacity will be generally higher. Settlement of single piles can be expected to be negligible.

Pile groups in cohesionless soils are considered to be acting as single piles as long as the piles are spaced at more than $2\frac{1}{2}$ to 3 times the pile diameter (Meyerhof). The bearing capacity of a pile group may then be taken as the number of piles times the safe bearing capacity of a single pile. The spacing of the piles of the present bridge is about 5 feet.

A bent with a pile spacing of less than $2\frac{1}{2}$ to 3 pile diameters is considered to be a long footing with width 'B' equal to the pile spacing. Assuming the piles to be at 3 diameters distance on centres the allowable load per pile of a bent would be of the order of 15 tons with differential settlement of $\frac{1}{4}$ inches being tolerated.

The river appears to be relatively slowly flowing and scour may be minimal. With piles founded at K1.65 in the river there is no probability of scour to endanger the pile foundation.

CONCLUSIONS:

The new Sag River Bridge is proposed to be founded on one foot diameter piles which will carry safely between 20 and 26 tons, each at 25' to 35' depth below the bridge deck, depending on the pile locations as shown on enclosure 4, below. The allowable load per pile for a spacing of three diameters, at which the foundation acts like a deep long footing, would be of the order of 15 tons for $\frac{1}{4}$ inch of differential settlement tolerated. Utilising the safe bearing values of 20 to 26 tons or more would only increase the settlement proportionately.

Scour is not expected to have effect on the foundation if the piles in the river penetrate to adequate bearing depth (K1.65 or about 35' below the bridge deck).



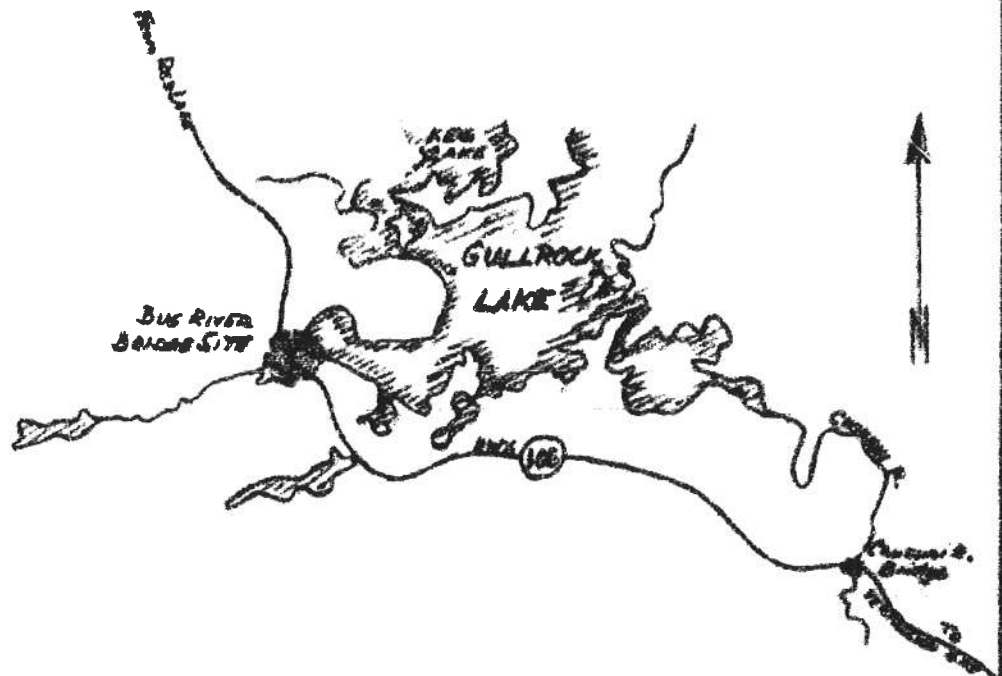
K. Tubbesing

K. TUBBESING, P. Eng.

HT/IN

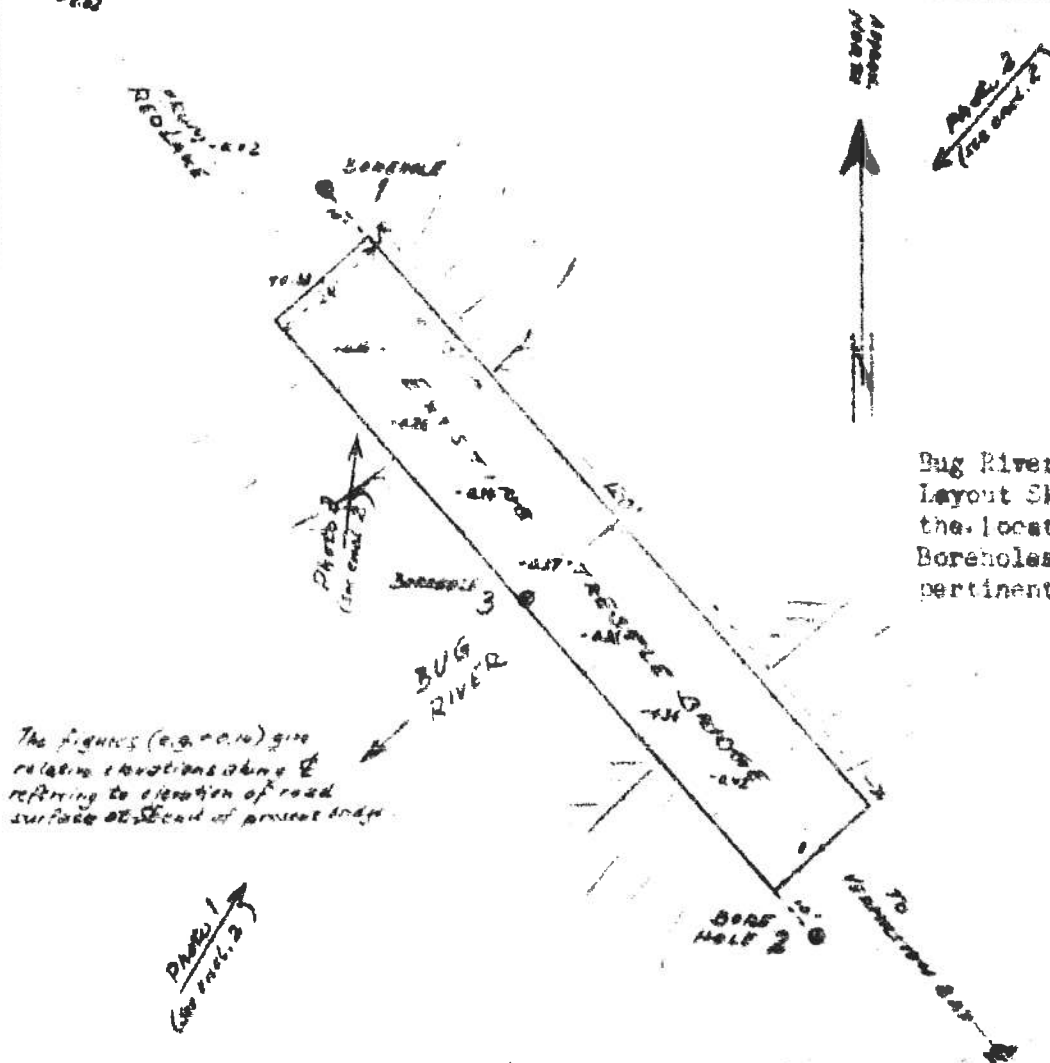
Prop. B. 1. Preliminary

Bug River Bridge.
Topographic Sketch
showing the location
of the site.



SCALE: 1" = 4 MILES
ADDED TOP MAP LAC SEUL

-0.02



Bug River Bridge.
Layout Sketch showing
the location of the
Boreholes and other
pertinent data.

The figures (e.g. 0.14) give
relative elevations above &
referring to elevation of road
surface at end of present bridge.

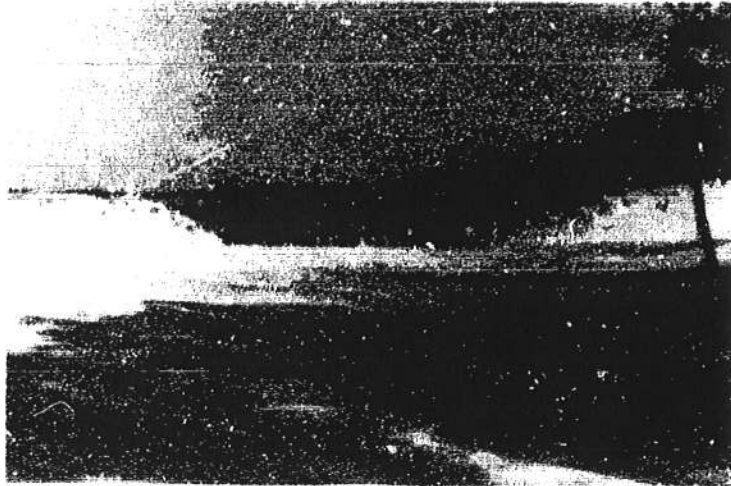


Fig.1. Present Bag
River Bridge
seen from south
west.



Fig.2. Present Bridge
seen from north
east.



Fig.3. Jacked up and
wedged founda-
tion of second
bent from north
west end of
bridge.

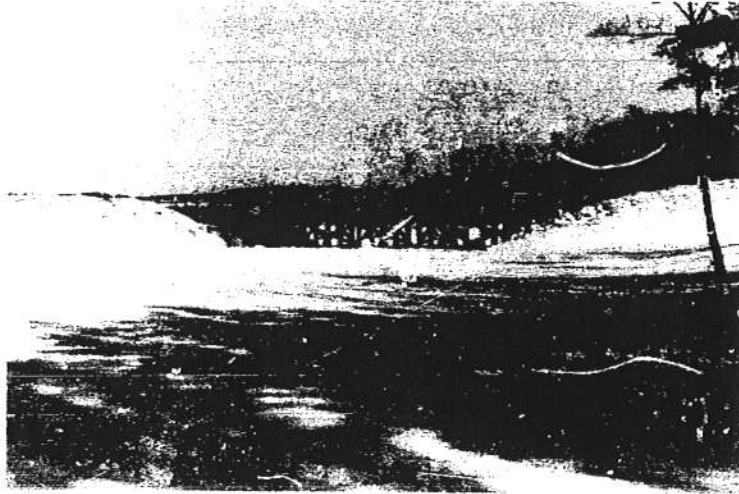


Fig.1. Present Bag
River Bridge
seen from south
west.

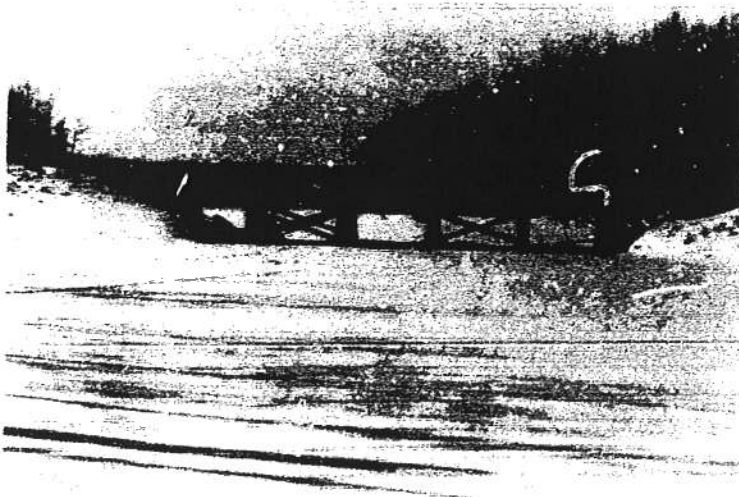


Fig.2. Present Bridge
seen from north
east.

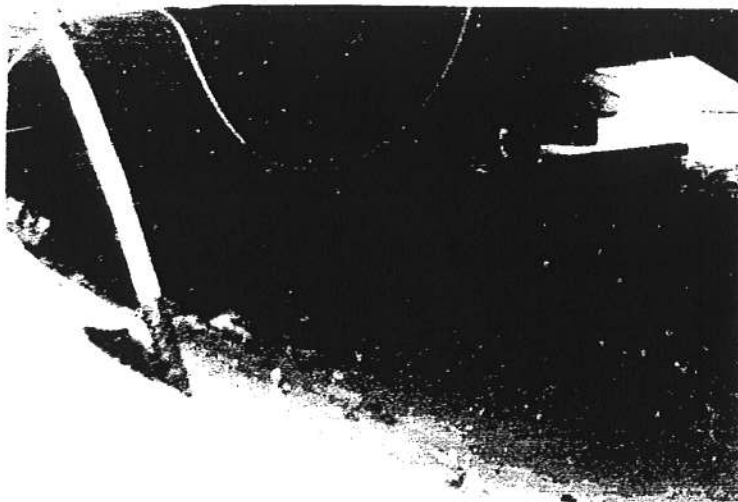


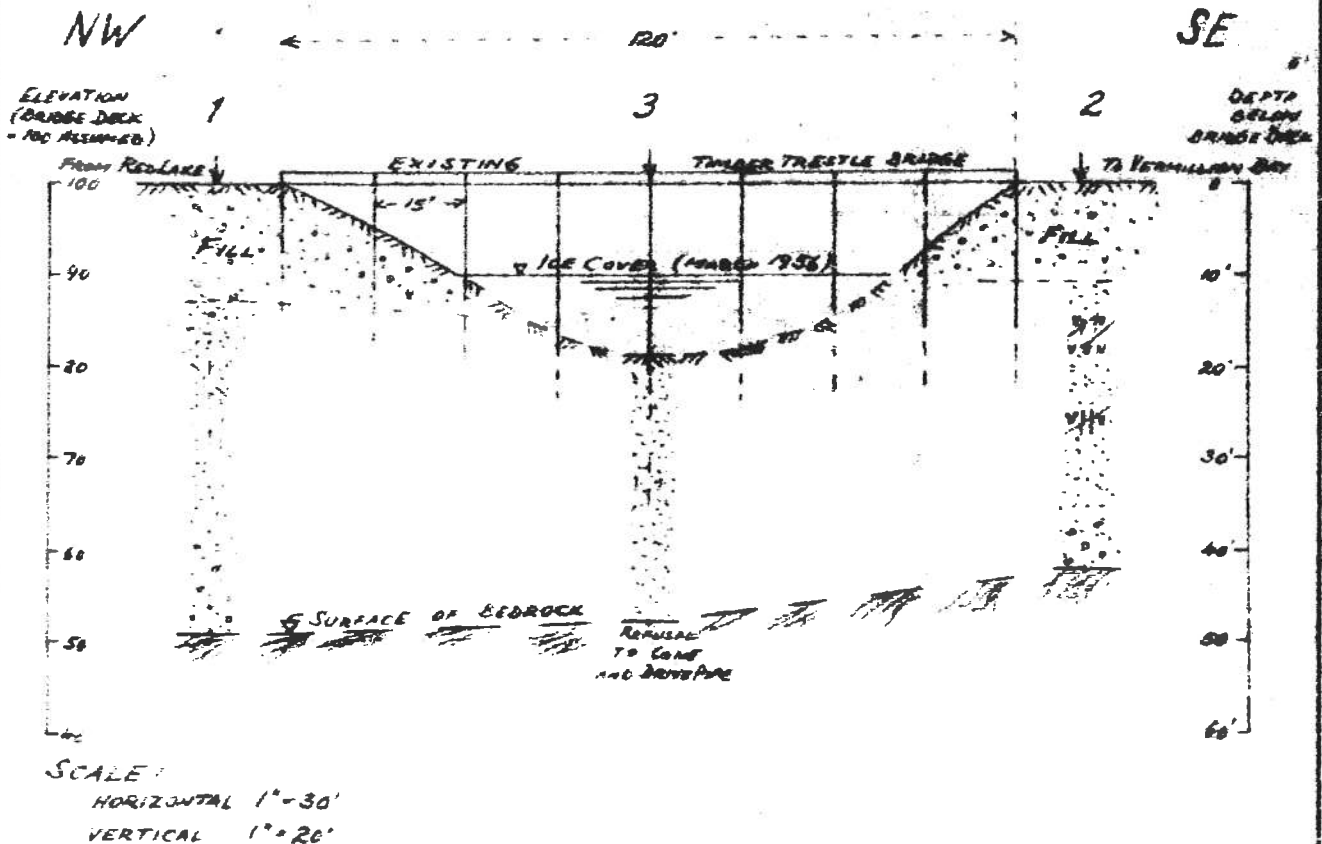
Fig.3. Jacked up and
wedged founda-
tion of second
bent from north
west end of
bridge.

Order No. S-500/T-262

Enclosure No. 3

Prep. By K. TUBGESING

Longitudinal Sectional Soil Profile
at the
Present Bug River Bridge



Prep. By K.T.

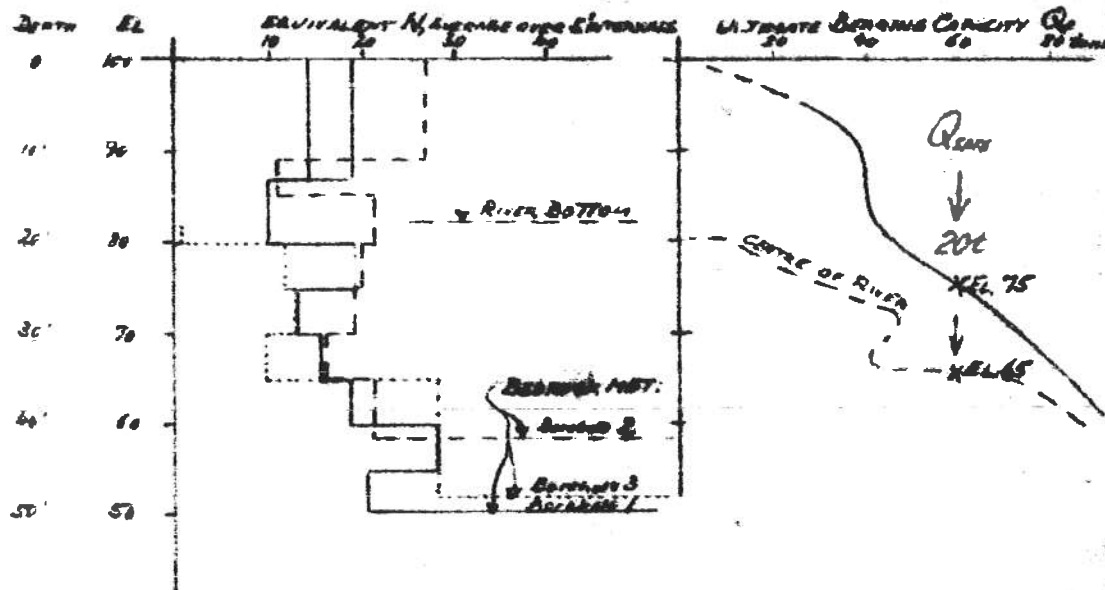
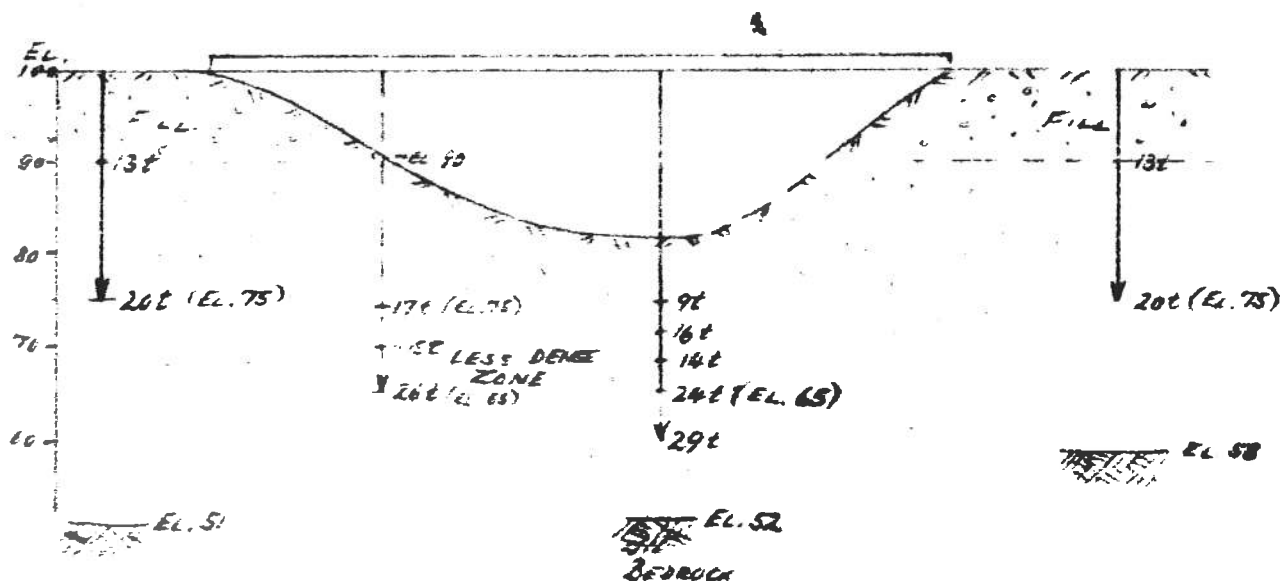


Diagram giving Average Penetration Resistance versus Elevation in the Testholes, and bearing capacities



Sectional Profile with safe bearing values (F.S.=3) at Various Point Elevations for some selected pile locations

Order No. S-500/T-262

Enclosure No. 5

RACEY McCALLUM AND ASSOCIATES LTD.

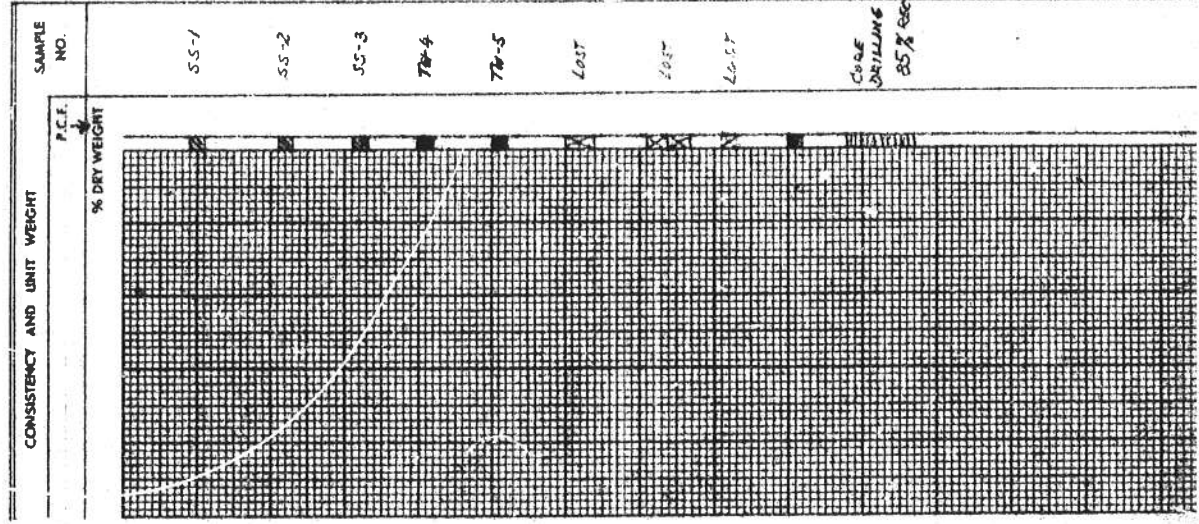
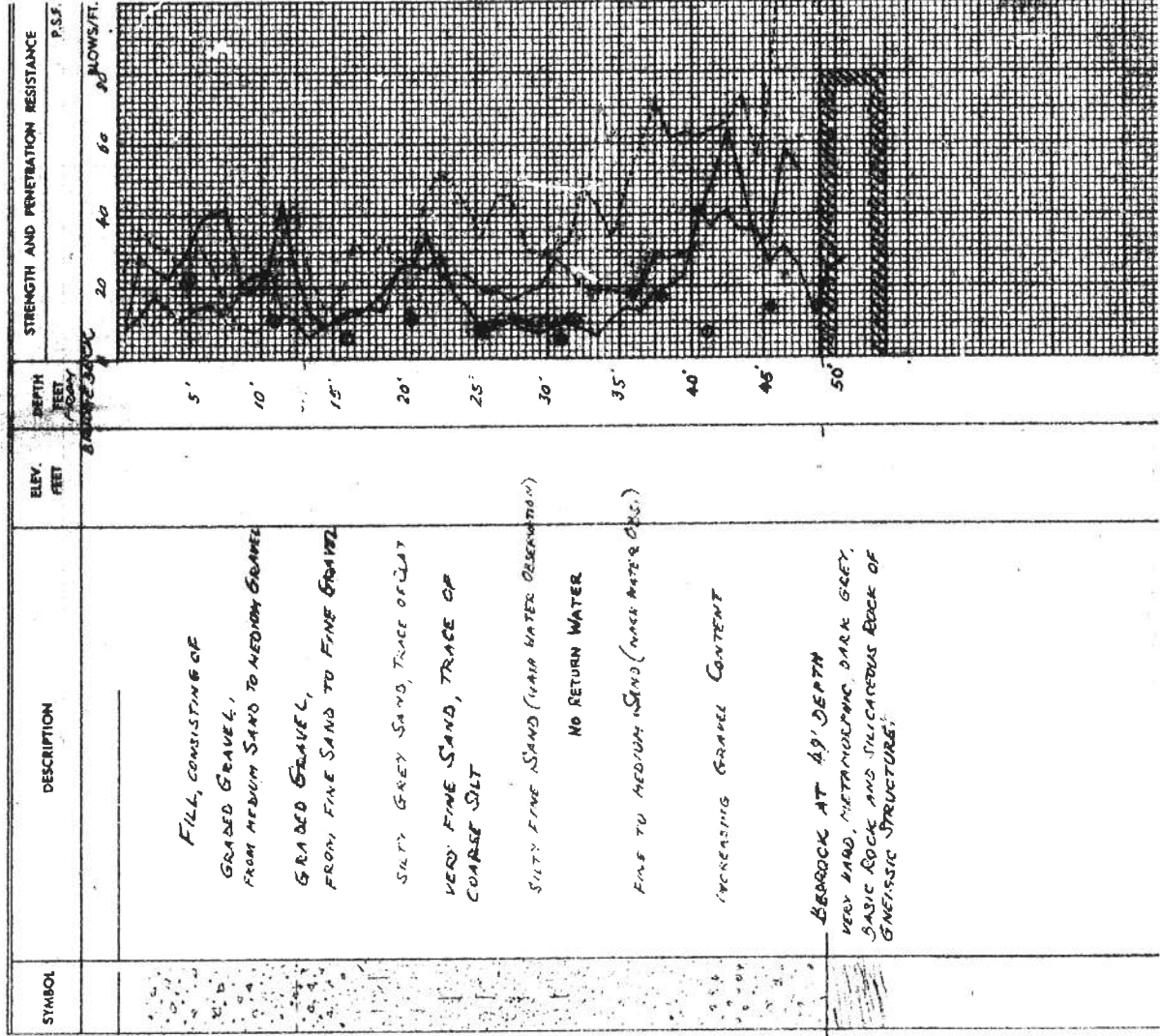
Foundation Engineering Division

Engineering Data Sheet for Borehole: 1

Project: PROPOSED NEW BUG RIVER BRIDGE
 Location: HWY 105, RED LAKE DISTRICT, ONT.
 Hole Location
 Hole Elevation and Datum: 496.4 94.1 1956
 Field Work Begun: APRIL 1956
 Field Supervision: R. CASS
 Driller: W. LINTON
 Prep: K. T.
 Checked: K. TUBBESING
 Date: APRIL 1964, 1956

LEGEND

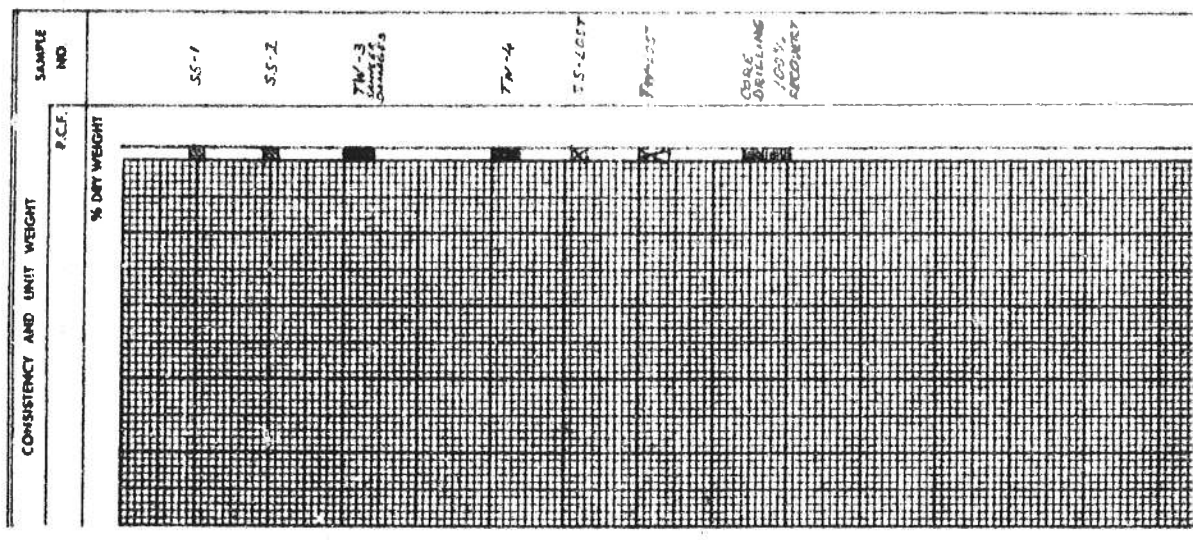
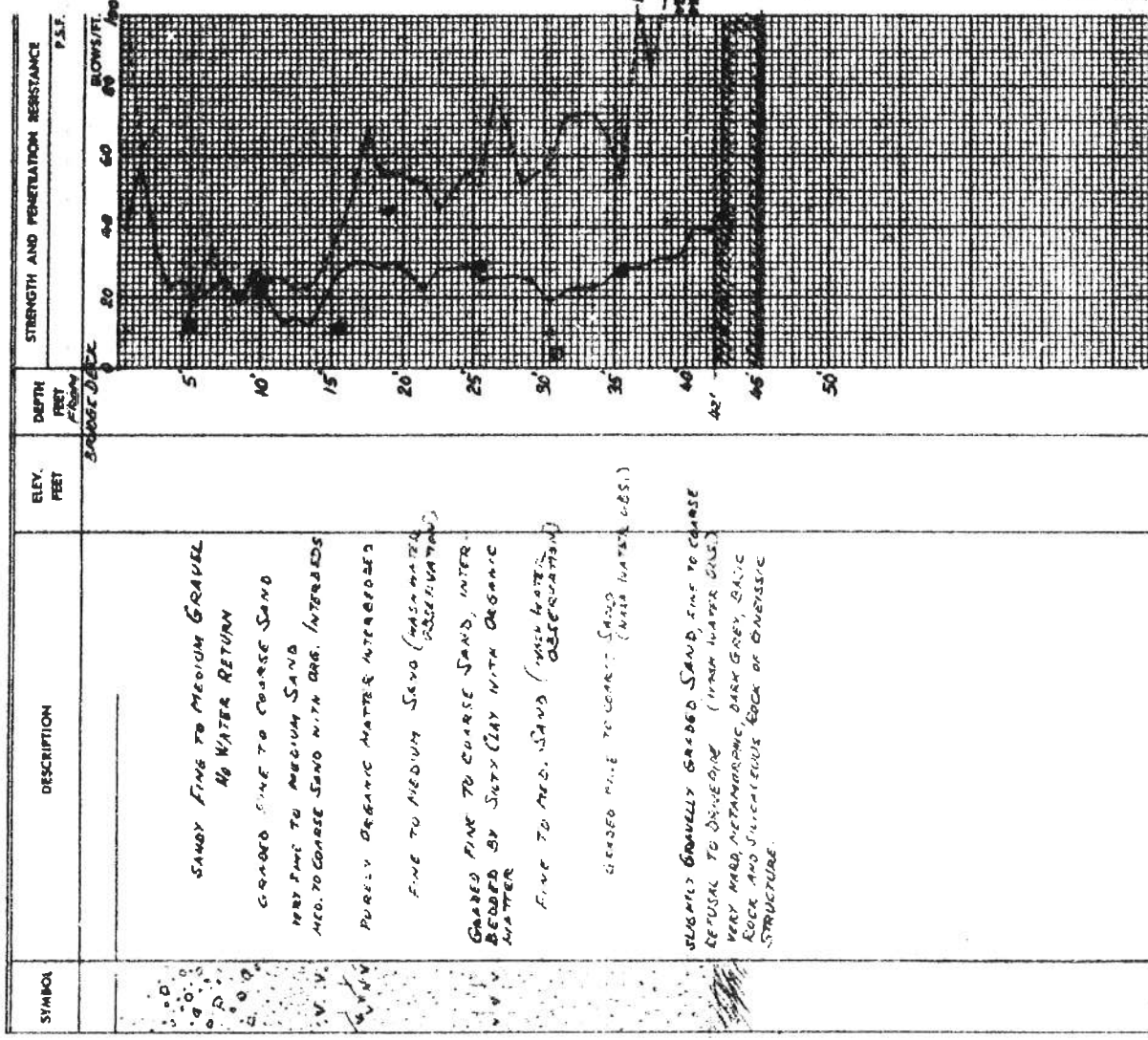
- Sampling Method
 - 2" Dia. split tube
 - 2" Shelby tube
- Penetration Resistance
 - 2" Split tube
 - 2" Dia. Cone
- Coring
 - Unconfined compression
 - Vane test and sensitivity
- Consistency
 - Natural moisture
 - Liquid limit
 - Plastic limit
 - Natural Unit Weight



RACEY MACCALLUM AND ASSOCIATES LTD.
Foundation Engineering Division
Engineering Data Sheet for Borehole: 2

Project: PROPOSED NEW BUG RIVER BRIDGE
Location: HWY. 105, RED LAKE DISTRICT, ONT.
Hole Location
Hole Elevation and Datum: April 12th 1956
Field Work Begun April 13th 1956
Ended April 13th 1956
Field Supervision: R. Cass
Driller: W. LINTON
Prep.: G.O. & A.T.
Checked: M. TUBBESING
Date: April 19th 1956

- LEGEND**
- Sampling Method
 - 2" Dia. split tube
 - 2" Shelby tube
 - Penetration Resistance
 - 2" Sym. cone
 - 2" Dia. Cone
 - Coring
 - Unconfined compression
 - Vane test and sensitivity
 - Consistency
 - Natural moisture
 - Liquid limit
 - Plastic limit
 - Natural Unit Weight
- (See Figure 1)



Order No. S-590/T-262

Enclosure No. 7

RACEY McCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 3

Project: PROPOSED NEW BIG RIVER BRIDGE
Location: HWY. 105, RED LAKE DISTRICT, ONT.

Hole Location

Hole Elevation and Datum

Field Work Begun APRIL 14TH, 1956

Ended APRIL 16TH, 1956

Date

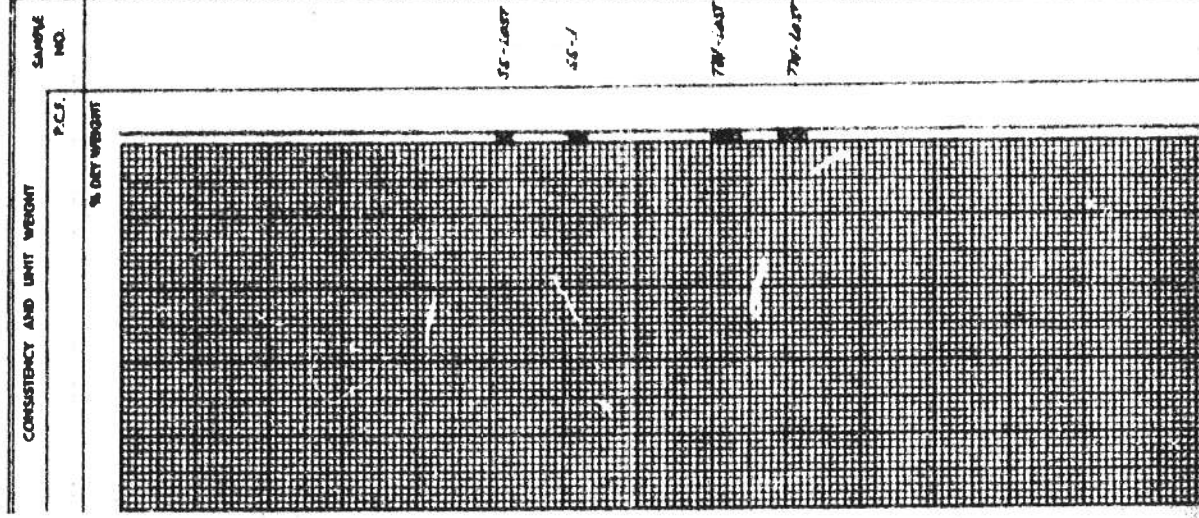
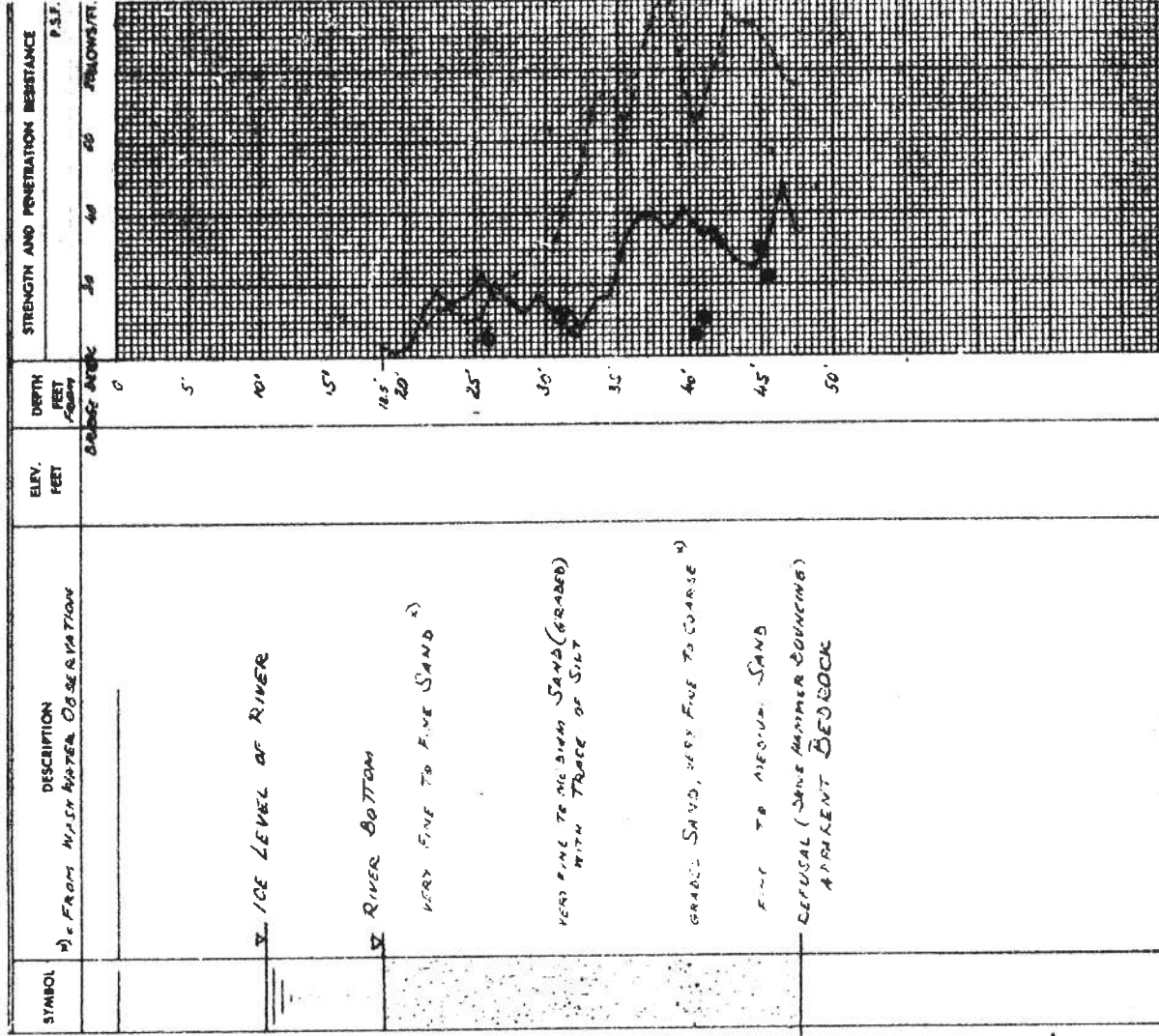
Field Supervisor R. CASS
Driller: Mc LINTON

Prep: K. T.

Checked: K. TUBBESING

NOTES

- Sampling Method
- 2" Dia. split tube
- 2" Split tube
- Penetration Resistance
- 2" Split tube
- 2" Dia. Cone
- Coring
- Strength
- Unconfined compression
- Vane test and sensitivity
- Consistency
- Natural moisture
- Liquid limit
- Plastic limit
- Natural Unit Weights



Appendix D

Site Photographs



Photograph 1 – Bug River Bridge



Photograph 2 – Lands north and west the Bug River Bridge



Photograph 3 –Bug River Bridge

Appendix E

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

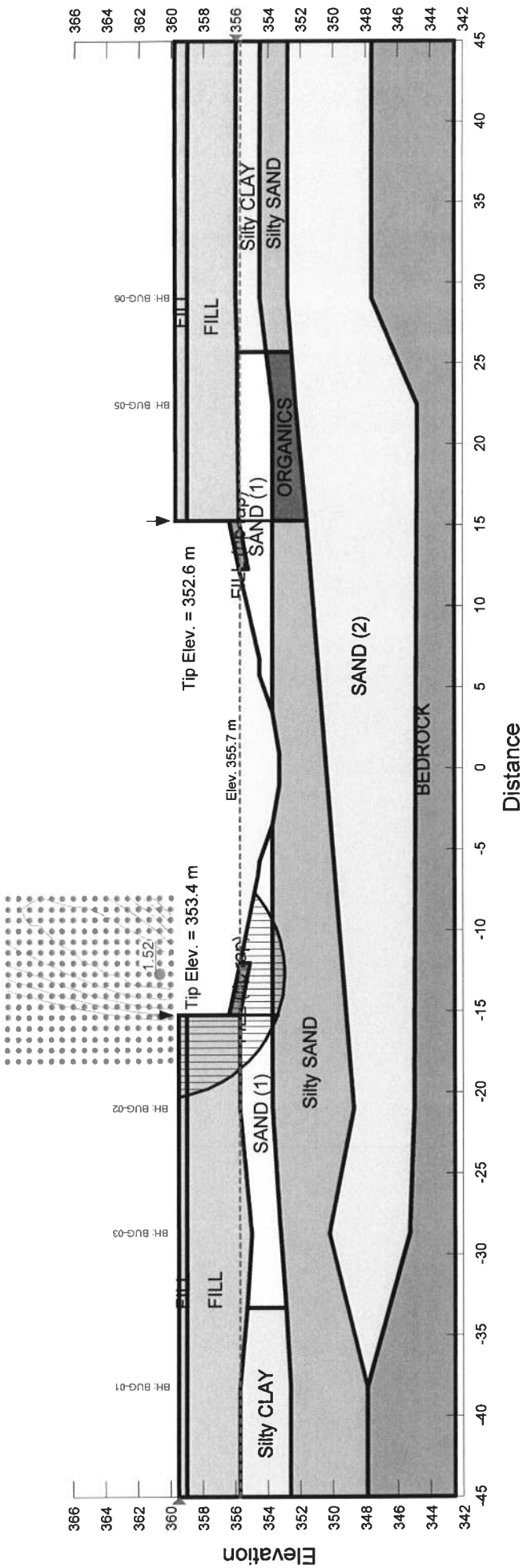
Driven Sheet Piles	Footings on Native Soil	Caissons	Driven H-Piles to Bedrock or Refusal
<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction. ii. Provides shoring and foundation elements in one operation. iii. Installation of piles could continue in freezing weather. iv. Potentially minimizes volume of excavation and roadway protection requirements. v. Minimizes potential for disturbance of streambed. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Unconventional design. ii. Cost of sheet piles. iii. Sheet pile lengths may vary due to variable depths of top of bedrock <p>FEASIBLE</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. <p>Disadvantages:</p> <ul style="list-style-type: none"> ii. Low available geotechnical resistance in native cohesionless deposits. iii. Dewatering will be required due to the high groundwater levels. iv. Potential disturbance of river during excavation. <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Construction of caissons could continue in freezing weather. ii. High geotechnical resistance available for units founded on bedrock. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher cost than spread footings ii. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons in cohesionless soils under the water table. iii. Potential difficulty in cleaning and inspecting bases. <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available by driving piles to achieve resistance on the bedrock or refusal. ii. Installation of piles could continue in freezing weather. iii. Foundation construction may require less volume of excavation than footings. iv. Readily installed. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. Pile lengths required to achieve design resistance may vary. <p>RECOMMENDED</p>

Appendix F

Slope Stability Outputs

FILL	20 kN/m³	0 kPa	32 °
FILL (rip rap)	19 kN/m³	0 kPa	40 °
Silty CLAY	18 kN/m³	0 kPa	27 °
SAND (1)	20 kN/m³	0 kPa	30 °
Silty SAND	20 kN/m³	0 kPa	30 °
ORGANICS	16 kN/m³	0 kPa	22 °
SAND (2)	20 kN/m³	0 kPa	30 °
BEDROCK			

Title: Bug River Bridge
 Comments: HWY 105, Kenora District, Ontario
 Description: Sheet Pile Wall
 Name: North Abutment, Embankment Height: 2.9m, Riverbank Slope: 4.8H:1V
 Last Solved Date: 9/28/2012, 11:36:19 AM
 Method: Morgenstern-Price
 Minimum Slip Surface Depth: 1 m
 Horz Seismic Load: 0



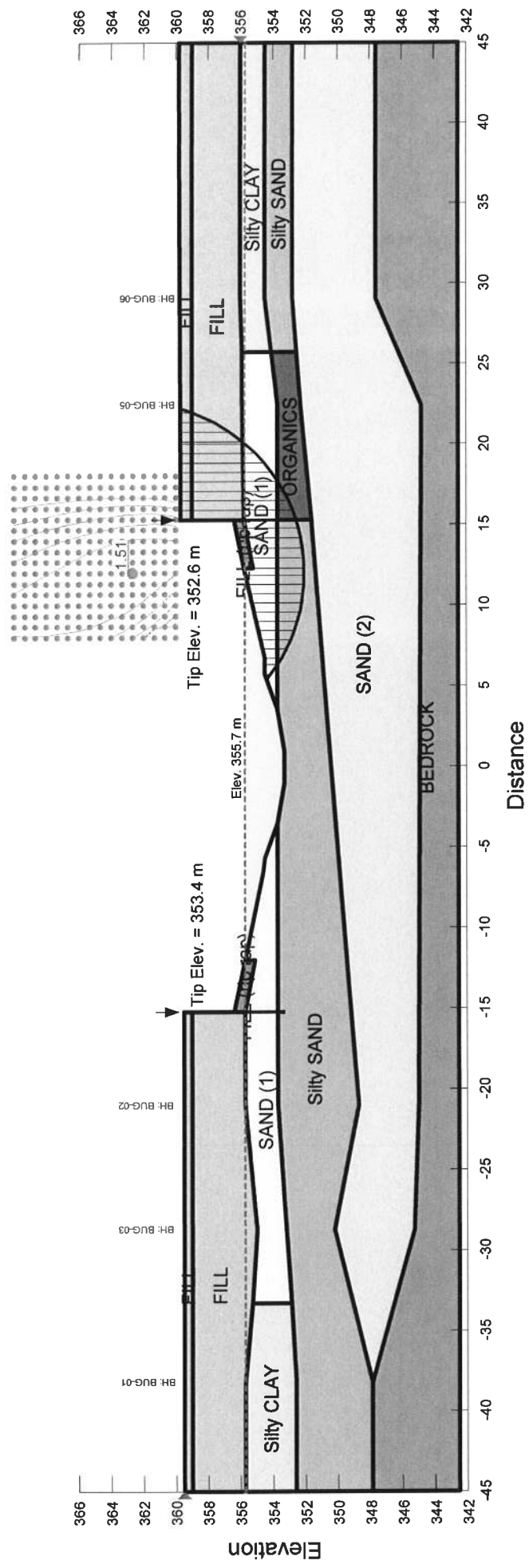
Directory: H:\1915308\40 NWR 11 Bridge 3 Culvert Rehab\Reports & Memos\Bug River Bridge\Analysis\Slope Stability\BugRiver_002.gsz

Figure 1

FILL	20 kN/m ³	0 kPa	32 °
FILL (rip rap)	19 kN/m ³	0 kPa	40 °
Silty CLAY	18 kN/m ³	0 kPa	27 °
SAND (1)	20 kN/m ³	0 kPa	30 °
Silty SAND	20 kN/m ³	0 kPa	30 °
ORGANICS	16 kN/m ³	0 kPa	22 °
SAND (2)	20 kN/m ³	0 kPa	30 °
BEDROCK			

Title: Bug River Bridge
 Comments: HWY 105, Kenora District, Ontario
 Description: Sheet Pile Wall
 Name: South Abutment, Embankment Height: 3.3m, Riverbank Slope: 4.5H:1V
 Last Solved Date: 9/28/2012, 11:36:47 AM

Method: Morgenstern-Price
 Minimum Slip Surface Depth: 1 m
 Horiz Seismic Load: 0



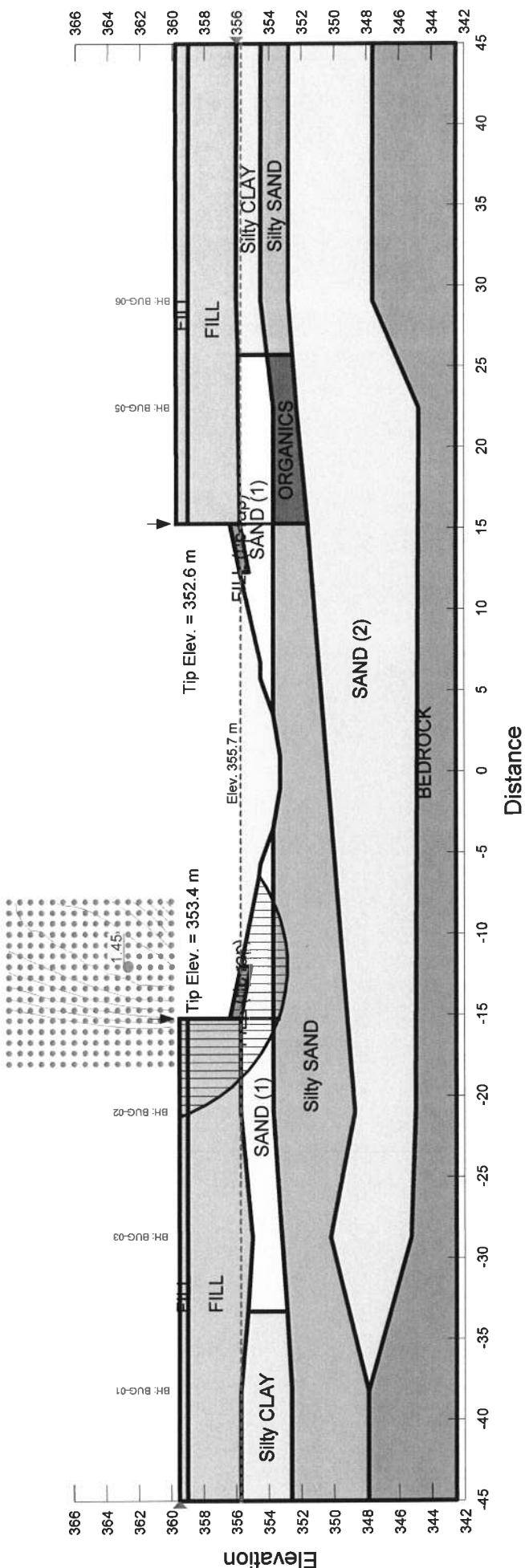
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Figure 2

Title: Bug River Bridge
Comments: HWY 105, Kenora District, Ontario
Description: Sheet Pile Wall
Name: North Abutment, Embankment Height: 2.9m, Riverbank Slope: 4.8H:1V, (Seismic)
Last Solved Date: 9/28/2012, 11:36:33 AM

Method: Morgenstern-Price
Minimum Slip Surface Depth: 1 m
Horz Seismic Load: 0.02

FILL	20 kN/m ³	0 kPa	32 °
FILL (rip rap)	19 kN/m ³	0 kPa	40 °
Silty CLAY	18 kN/m ³	0 kPa	27 °
SAND (1)	20 kN/m ³	0 kPa	30 °
Silty SAND	20 kN/m ³	0 kPa	30 °
ORGANICS	16 kN/m ³	0 kPa	22 °
SAND (2)	20 kN/m ³	0 kPa	30 °
BEDROCK			



Directory: H:\19\5308\40 NWR 11 Bridge 3 Culvert Rehabs\Reports & Memos\Bug River Bridge\Analysis\Slope Stability\BugRiver_002.gsz

Figure 3

Title: Bug River Bridge

Comments: HWY 105, Kenora District, Ontario

Description: Sheet Pile Wall

Name: South Abutment, Embankment Height: 3.3m, Riverbank Slope: 4.5H:1V, (Seismic)

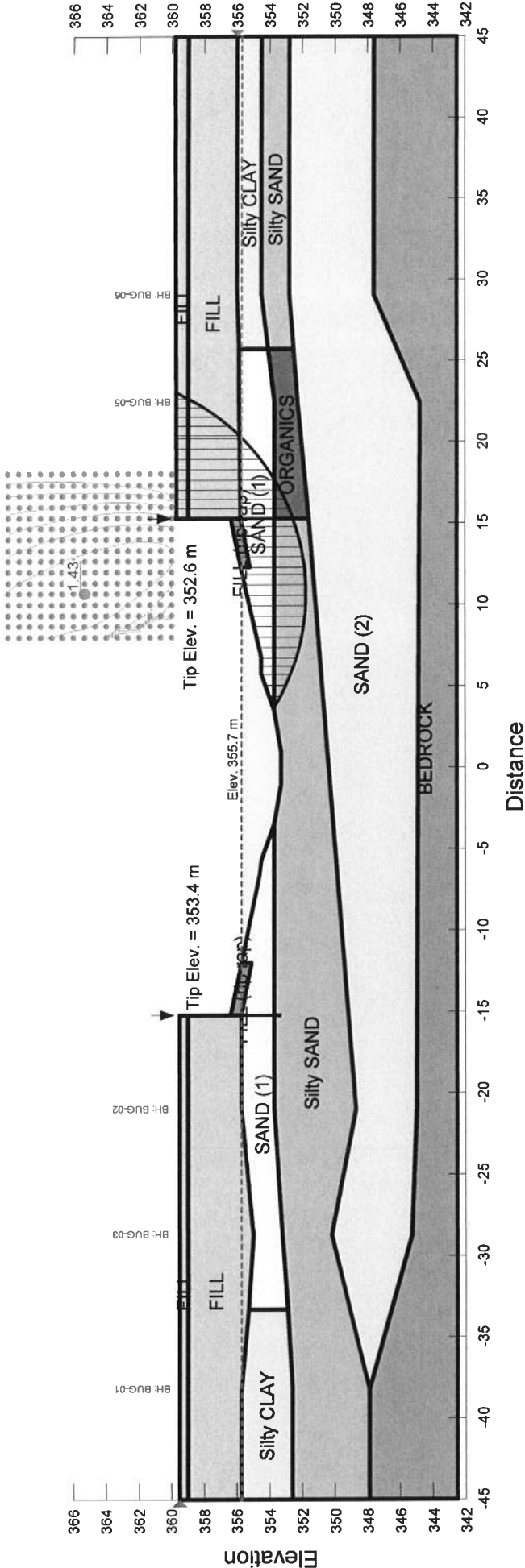
Last Solved Date: 9/28/2012, 11:37:01 AM

Method: Morgenstern-Price

Minimum Slip Surface Depth: 1 m

Horz Seismic Load: 0.02

FILL	20 kN/m ³	0 kPa	32 °
FILL (rip rap)	19 kN/m ³	0 kPa	40 °
Silty CLAY	18 kN/m ³	0 kPa	27 °
SAND (1)	20 kN/m ³	0 kPa	30 °
Silty SAND	20 kN/m ³	0 kPa	30 °
ORGANICS	16 kN/m ³	0 kPa	22 °
SAND (2)	20 kN/m ³	0 kPa	30 °
BEDROCK			



Directory: H:\19\5308\40 NWR 11 Bridge 3 Culvert Rehabs\Reports & Memos\Bug River Bridge\Analysis\Slope Stability\BugRiver_002.gsz

Figure 4

Appendix G

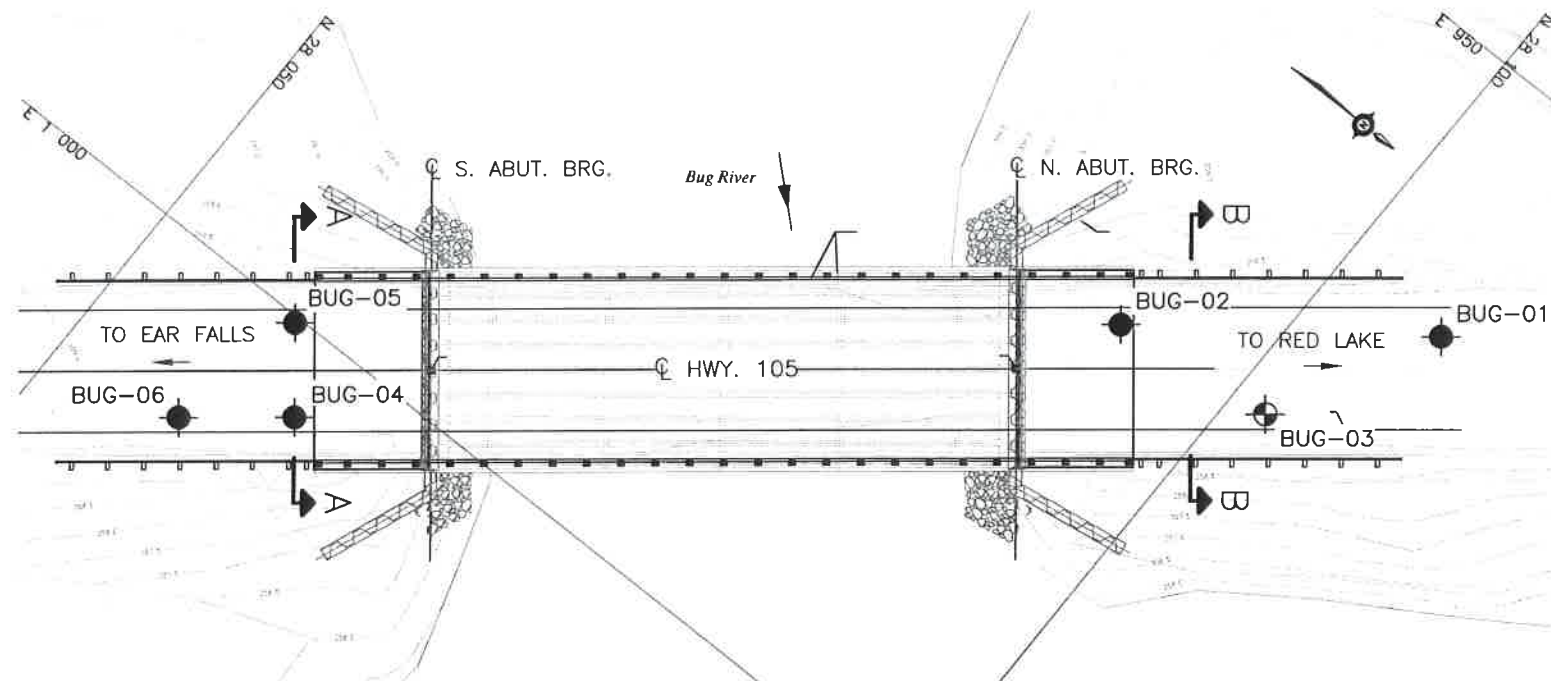
List of SPs and OPSS, and Suggested Text for NSSP

1. List of Special Provisions and OPSS Documents Referenced in this Report

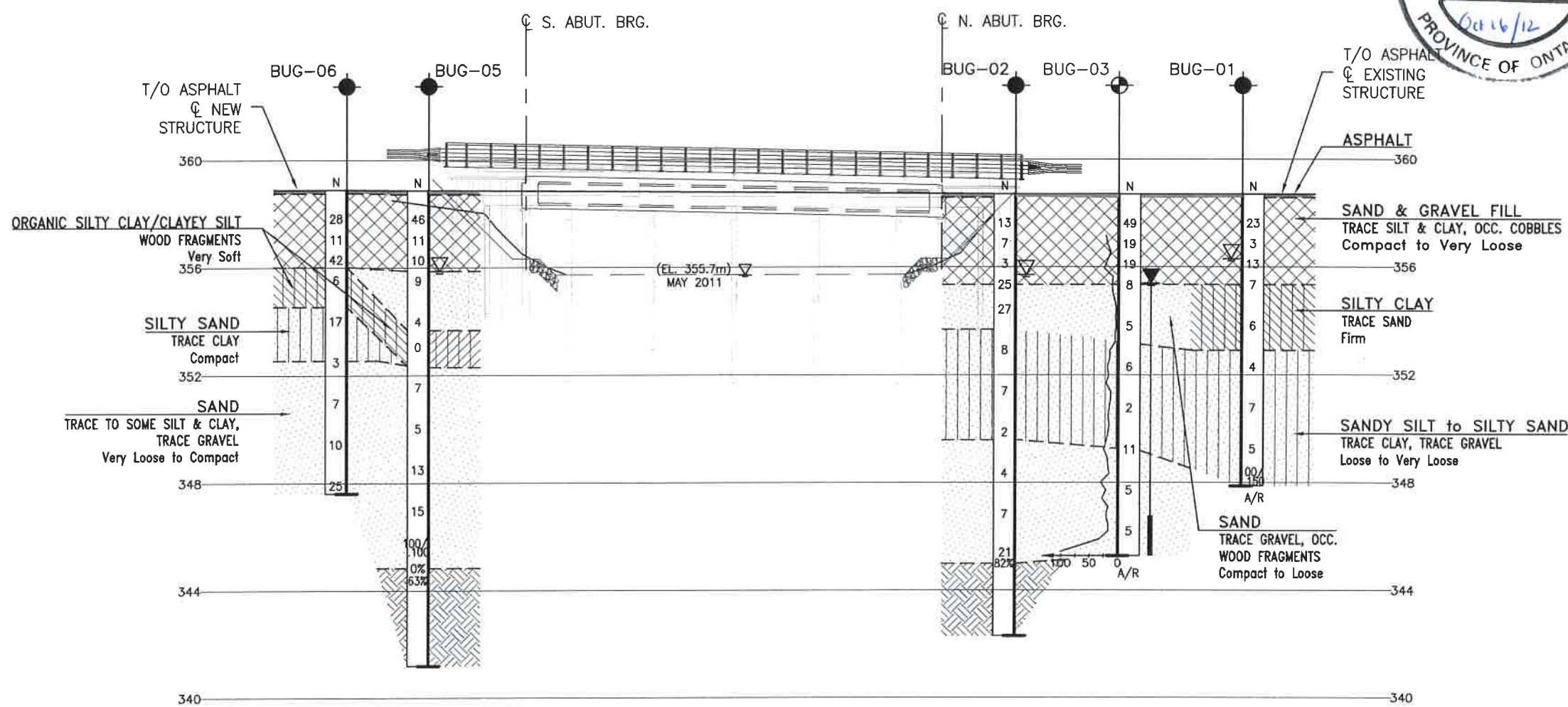
- OPSS 903
- OPSS 804
- OPSS 902
- OPSD 3101.150.
- Special Provision 110S13 “Amendment to OPSS 1010”.
- OPSS 539
- OPSS 501

Appendix H

Borehole Locations and Soil Strata Drawings



PLAN
SCALE 1:400



PROFILE ALONG ϕ HWY. 105

SCALE 1:400
SCALE 1:200

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

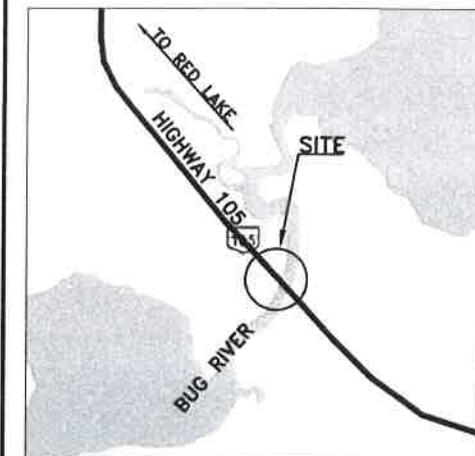


CONT No 2012-6015
WP No 6942-10-01

BUG RIVER BRIDGE
REPLACEMENT
HIGHWAY 105
BOREHOLE LOCATIONS AND SOIL STRATA

GENIVAR

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

	Borehole
	Borehole and Cone
	Blows /0.3m (Std Pen Test, 475J/blow)
	Blows /0.3m (60' Cone, 475J/blow)
	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
	90% Rock Quality Designation (RQD)
	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
BUG-01	358.7	281 06.6	963.4
BUG-02	358.7	280 93.1	973.3
BUG-03	358.7	281 02.0	972.3
BUG-04	358.8	280 62.1	1 004.2
BUG-05	358.9	280 59.0	1 000.3
BUG-06	358.9	280 57.3	1 008.0

NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 52K-8

DATE	BY	DESCRIPTION
DESIGN	LRB	CHK LRB
DRAWN	AN	CHK
DATE	OCT. 2012	
DWG	1	

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No 2012-6015
WP No 6942-10-01

BUG RIVER BRIDGE
REPLACEMENT
HIGHWAY 105
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

GENIVAR



THURBER ENGINEERING LTD.



**KEYPLAN
LEGEND**

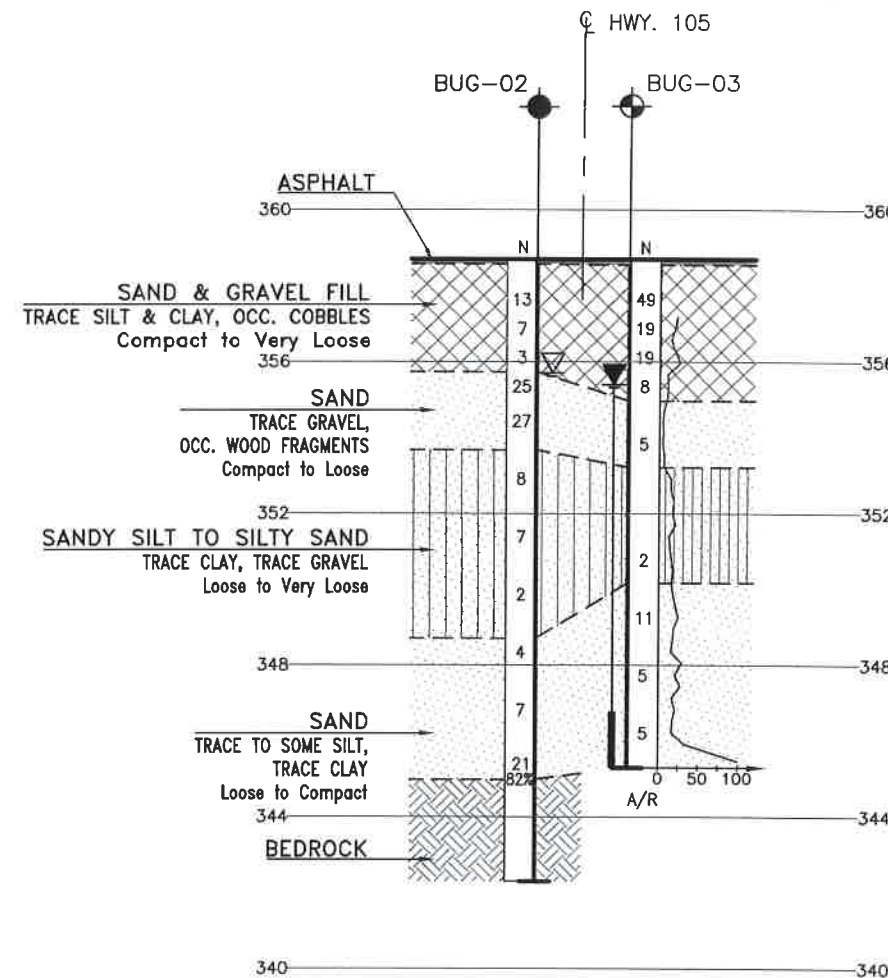
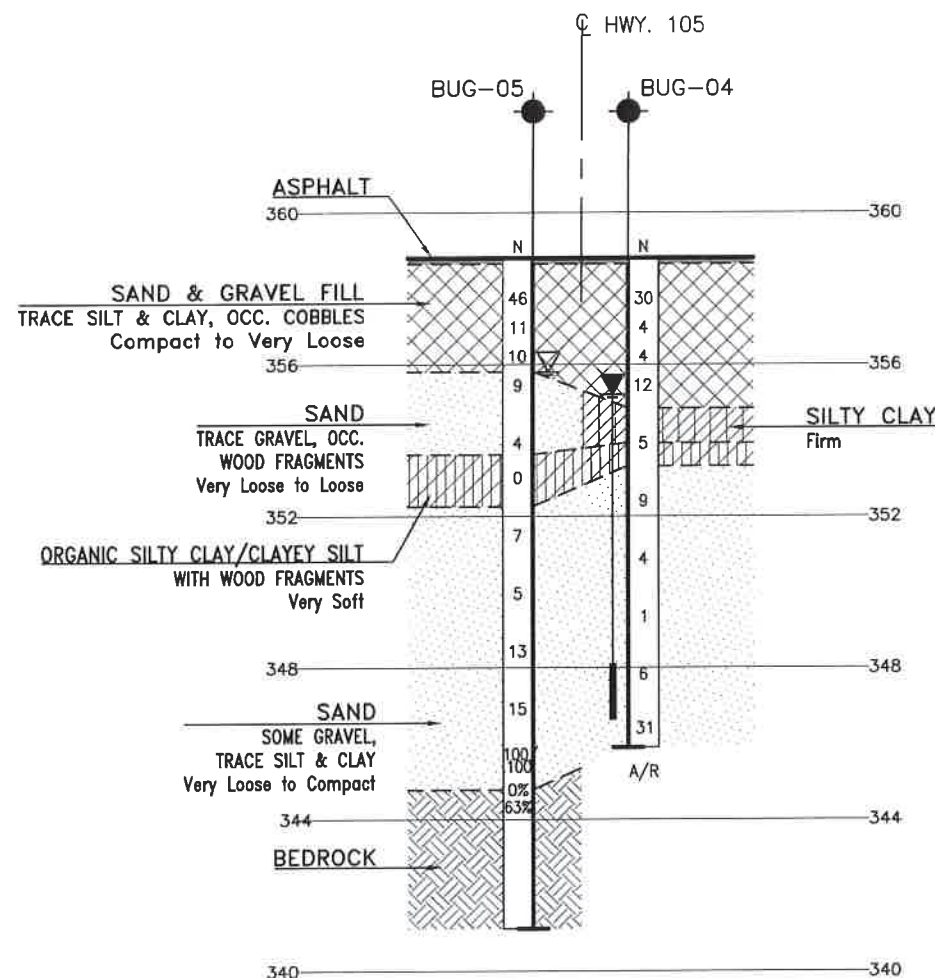
●	Borehole
⊕	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
W	Water Level
HA	Head Artesian Water
P	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
BUG-01	358.7	281 06.6	963.4
BUG-02	358.7	280 93.1	973.3
BUG-03	358.7	281 02.0	972.3
BUG-04	358.8	280 62.1	1 004.2
BUG-05	358.9	280 59.0	1 000.3
BUG-06	358.9	280 57.3	1 008.0

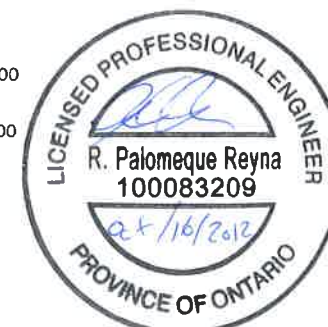
-NOTES-

- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCREs No. 52K-8



H 1:400
V 1:200

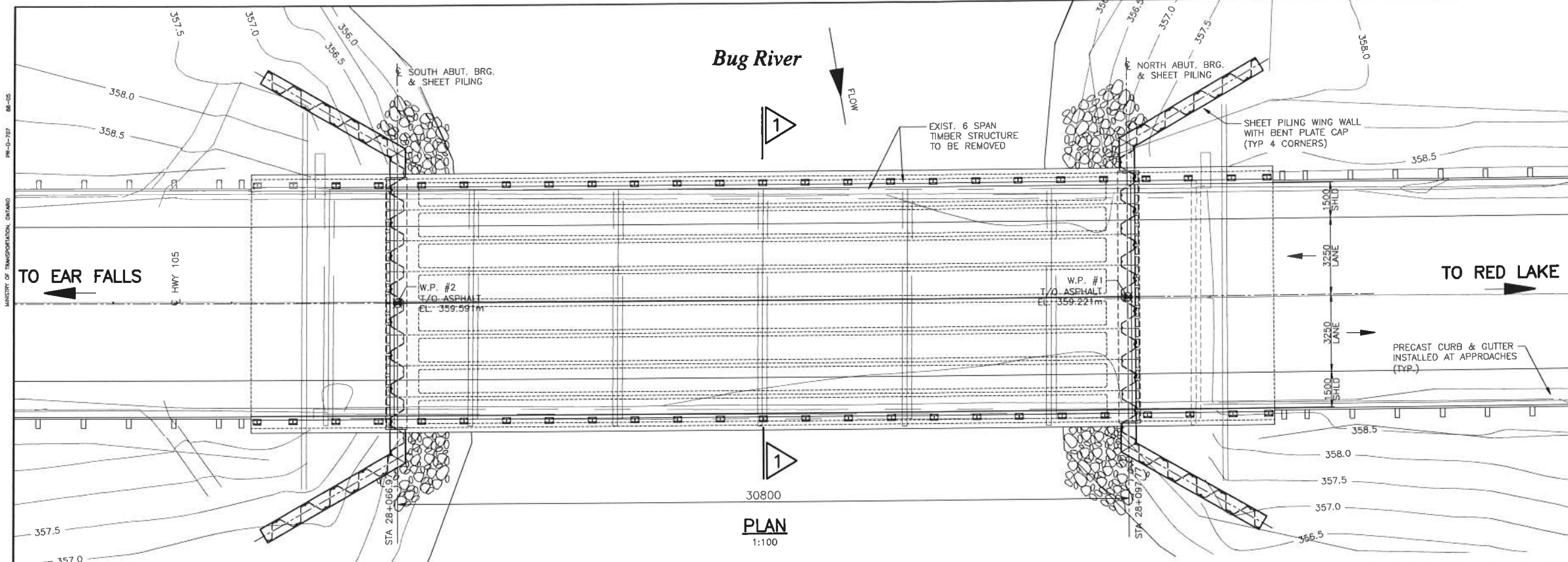


REVISIONS	DATE	BY	DESCRIPTION
DESIGN	LRB	CHK	LRB
DRAWN	AN	CHK	SITE
			STRUCT
			DWG 2
			DATE OCT. 2012

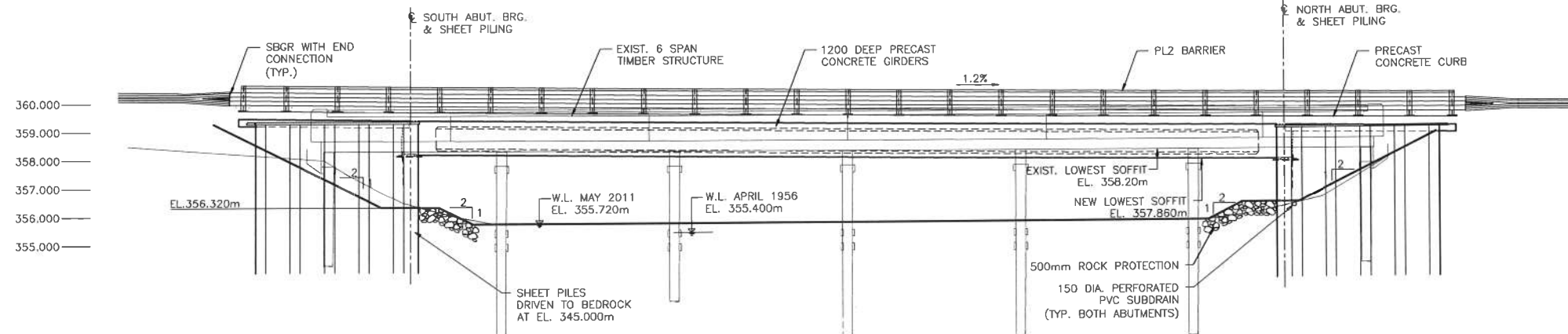
Appendix I

General Arrangement Drawing provided by Genivar

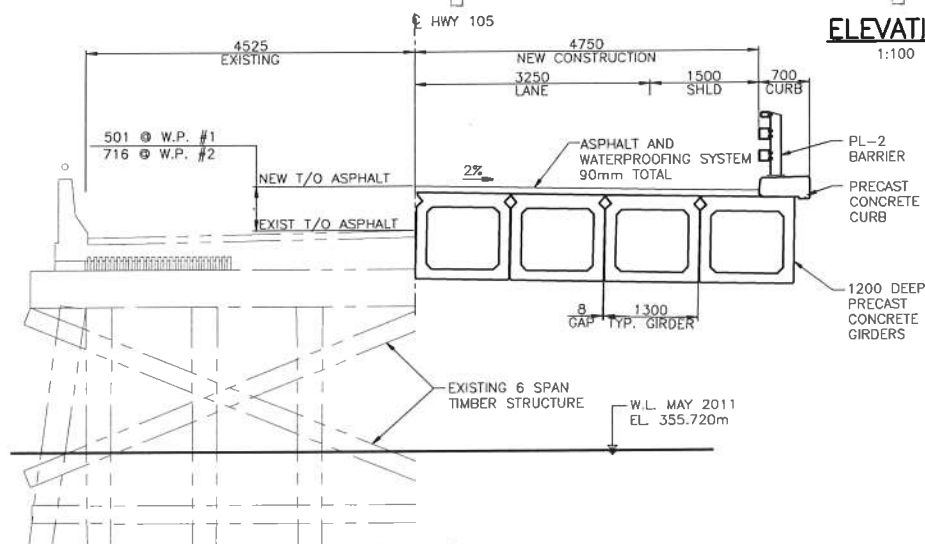
CAD FILE LOCATION AND NAME: P:\Transportation\MOY\2011\111-53428-00 14 Structures\Working Dega\JSM WORKING\1 BUG RIVER CA.dwg
MODIFIED: 9/21/2012 4:18:54 PM BY: JUL MAGBANUA
DATE PLOTTED: 9/21/2012 4:19:59 PM BY: JUL MAGBANUA



PLAN
1:100



ELEVATION
1:100

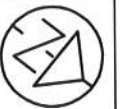


1:50

CONT No
WP No

BUG RIVER BRIDGE

GENERAL ARRANGEMENT



SHEET



GENERAL NOTES

CLASS OF CONCRETE

ALL PRECAST CONCRETE..... 60 MPa

CLEAR COVER TO REINFORCEMENT

PRECAST CONC. APPROACH PANEL - TOP & SIDES - 35±10
PRECAST CONC. FRP TENDONS - 50±10

GLASS FIBRE REINFORCED POLYMER (GFRP) BARS

1. GLASS FIBRE REINFORCED POLYMER BARS SHALL BE GRADE I, GRADE II OR GRADE III AS SPECIFIED IN THE CONTRACT DRAWINGS. THE NOMINAL DIAMETER, TENSILE MODULUS OF ELASTICITY AND GUARANTEED MINIMUM TENSILE STRENGTH SHALL BE AS SPECIFIED IN THE CONTRACT DOCUMENTS. BAR MARKS WITH THE PREFIX GI DENOTE GRADE I GLASS FIBRE REINFORCED POLYMER BARS. BAR MARKS WITH THE PREFIX GII DENOTE GRADE II GLASS FIBRE REINFORCED POLYMER BARS. BAR MARKS WITH THE PREFIX GIII DENOTE GRADE III GLASS FIBRE REINFORCED POLYMER BARS.

CONSTRUCTION NOTES

1. THE CONSTRUCTION SHALL BE COMPLETED IN STAGES.
2. THE PRECAST CONCRETE GIRDERS AND CURBS SHALL BE PRESTRESSED MEMBERS.
3. TRAFFIC BARRIERS SHALL BE PERFORMANCE LEVEL PL2.
4. THE CONTRACTOR IS RESPONSIBLE FOR MAINTAINING THE STABILITY OF THE EXISTING STRUCTURE THROUGHOUT CONSTRUCTION.
5. ACCESS TO THE WORK AREA IS LIMITED TO THE EXISTING ROADBED AREA BEHIND THE TEMPORARY CONCRETE BARRIERS. THE CONTRACTOR IS NOT PERMITTED TO WIDEN THE ROADWAY FOR CONSTRUCTION EQUIPMENT.
6. THE CONTRACTOR IS ADVISED NOT TO RELY ON THE WATER LEVEL SHOWN ON THE DRAWINGS. THE WATER LEVEL IS SUBJECT TO VARIATIONS.
7. BACKFILL SHALL NOT BE PLACED BEHIND THE ABUTMENTS UNTIL THE PRESTRESSED CONCRETE GIRDERS ARE IN PLACE AND GROUT HAS REACHED 70% OF STRENGTH.

LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATION & SOIL STRATA 1
3. BOREHOLE LOCATION & SOIL STRATA 2
4. CONSTRUCTION STAGING
5. DEMOLITION & ENVIRONMENTAL
6. ABUTMENT & PILING LAYOUT
7. ABUTMENT & PILING DETAILS
8. PRECAST CONCRETE GIRDER LAYOUT
9. PRECAST CONCRETE GIRDER DETAILS
10. PRECAST CURB LAYOUT & DETAILS
11. PRECAST CURB DETAILS
12. PRECAST CURB & GUTTER DETAILS
13. RAILING LAYOUT & DETAILS
14. PRECAST CONCRETE APPROACH SLABS
15. MISCELLANEOUS DETAILS & STANDARDS

WORKING POINTS-LOCAL CO-ORDINATES		
W.P. #	NORTHING	EASTING
1	28 066.204	997.815
2	28 090.268	978.591

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS		DESCRIPTION
DESIGN	AAI	CHK PAS [CODE CHBDC-2010] [LOAD CL-625-ONT] [DATE MAY 2012]
DRAWN	CSN	CHK DCR [SITE 41N-002] [STRUCT] [SCHEME] [DWG] 1